reinforced brick masonry lateral force design

REINFORCED BRICK MASONRY and LATERAL FORCE DESIGN

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PREFACE

In writing this book the authors have attempted to present to the design profession and to the construction industry, in as concise form as possible, data on the performance of masonry, both reinforced and unreinforced, recommended design and construction procedures based upon these data, and a review of the currently accepted design criteria, particularly those relating to lateral forces resulting from wind, earthquake or blast.

While this book is primarily for the use of architects and engineers who are experienced in design and construction, it is hoped that it will be of value also to undergraduates of architectural and engineering colleges and to the young graduates of such colleges just embarking upon their professional careers.

Chapters 1 to 8, inclusive, cover the subjects listed above and the recommendations affecting design are based on the assumption that the reader has a fundamental knowledge of the principles of mechanics and stress analysis.

Examples are given in the Appendix in which various design formulae are applied to specific problems, and design tables are included similar to those found in most handbooks. These latter should be an aid to both the experienced designer and the beginner.

In the preparation of this book, liberal use has been made of the publications of the Structural Clay Products Institute, particularly Brick Engineering and Tile Engineering, and the authors acknowledge the substantial contributions of Leslie J. Reardon, Professor of Applied Mechanics, Case School of Technology, and Edwin F. Wanner, Chief Engineer, Natco Corporation, the co-authors of Brick Engineering and Tile Engineering respectively. Acknowledgment is also made of the constructive suggestions of the SCPI Regional Directors and of the assistance of the staffs of the authors, particularly Sarah A. Hardy and Donald M. Teixeira, assistants to Plummer and Blume, respectively.

WASHINGTON, D. C.
November, 1953

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CHAPTER 1

HISTORY AND DESCRIPTION

101. INTRODUCTION

Reinforced brick masonry, as the name implies, consists of brick masonry in which steel reinforcement is embedded and so placed that the masonry will have greatly increased resistance to forces which produce tensile, shearing and compressive stresses. The principles of reinforced brick masonry design are the same as those commonly accepted for reinforced concrete, and similar formulae may be used.

Since reinforced masonry is designed to resist bending as well as compression, it is essential that all joints in the masonry be completely filled. The method recommended for accomplishing this is to fill all interior joints with grout which is obtained by adding sufficient water to the mortar to give it a fluid consistency.

Most writers on reinforced brick masonry point out that it is not a new type of construction and refer to its use by Marc Isambard Brunel in 1825. While it is true that reinforced brick masonry was used by many builders during the last century, these builders were, in the main, those who had a "feeling" for materials and built their structures, based upon their experience, more as an art than from a rational design.

Prior to 1880, a few attempts were made to develop design formulae; however, the performance of composite steel and masonry flexural members was not clearly understood at that time and many investigators attributed the strength of the construction primarily to the adhesive properties of the masonry. In fact, most of the early tests were designed to demonstrate the increased strength obtainable through the use of the "new Portland Cement" in mortar instead of the hydraulic limes and natural cements formerly used. These experiments were financed by cement manufacturers and the results undoubtedly aided materially in introducing portland cement to European engineers.

With the invention of reinforced concrete, variously attributed to Lambot in 1850 or Monier in 1861, extensive experimental investigations were made of this type of construction and, as the volume of data on the performance of reinforced concrete beams and other structural members accumulated, design procedures were developed which were the forerunners of the rational design methods for reinforced masonry construction.

In the United States, the first Joint Committee on Standard Specifications for Concrete and Reinforced Concrete was organized in 1904 which, quoting from the 1940 report of the Joint Committee, "marks approximately the beginning of reinforced concrete construction in this country. At that time the concrete construction was primarily in the hands of a few specialists whose methods varied with their experience."

The first joint committee submitted its final report in 1916. This report was devoted primarily to design procedures and served as a basis for building code and specification requirements throughout the country. Its recommendations were also adopted widely by authors of text books and engineering handbooks on reinforced concrete.

During the period 1880 to 1920, little use seems to have been made of reinforced brick masonry and experimental investigation of this type of construction appears to have been practically discontinued.

About 1920, as a result of the "high cost" of structural steel and reinforced concrete construction, the Government of India undertook an extensive investigation of the performance of reinforced brick masonry structural members. This investigation extended over a period of about two years and the results are reported in Technical Paper No. 38 of the Public Works Department, Government of India, by Under Secretary A. Brebner. This report stimulated an interest in reinforced brick masonry in this country on the part of manufacturers and structural engineers, and both the Brick Manufacturers Association of America and the National Brick Manufacturers Research Foundation sponsored tests on reinforced brick masonry structures to obtain data from which to develop design formulae and to establish working stresses.

The latter appointed a Committee on Reinforced Brick Masonry "for the purpose of stimulating interest in reinforced brick masonry design, initiating and correlating research work to develop the facts necessary to establish the flexural theories involved, and translating such theories into practical design formulae for the use of architects and designing engineers."

This committee, Edw. E. Krauss, Chairman, and Judson Vodges, Secretary, submitted eight reports during the period from October 1931 to February 1933; all of which are listed in the Bibliography of References.

The investigations, sponsored by the Brick Manufacturers Association of America and the National Brick Manufacturers Research Foundation, form the basis for the currently accepted design and construction practices for reinforced brick masonry. During the past 20 years many additional data have been obtained through research, construction experience and the performance of structures, and these have been used to refine both design and construction procedures.

While brick masonry is one of the oldest forms of building construction and reinforcement has been used to strengthen masonry at various times during the past century, in the modern sense reinforced brick masonry in the United States is a relatively new type of construction, requiring new design procedures and construction methods. These have been developed during the past 30 years from experimental investigations and through the construction of hundreds of buildings which have demonstrated the practicability and economy of the construction and whose performances have confirmed the soundness of the principles of design.

102. HISTORY

Report No. 5, "Reinforced Brick Masonry: History, Summary of Tests, Structures Erected, and Bibliography", of the Committee on Reinforced Brick Masonry of the National Brick Manufacturers Research Foundation is an exhaustive treatment of the subject and it is not the purpose of this book to repeat the interesting historical information included in the report. What follows is a brief summary of this material to indicate the recurring interest in reinforced brick masonry construction during more than a century.

Marc Isambard Brunel, once Chief Engineer of the City of New York and later knighted by Queen Victoria, is credited with the discovery of reinforced masonry about 140 years ago.

He first proposed the use of reinforced brick masonry in 1813 as a means of strengthening a chimney then under construction; however, it was in con-

nection with the building of the Thames Tunnel in 1825 that he made his first major application of its principles. As a part of the construction of this tunnel, two brick shafts were built, each 30 in. thick, 50 ft. in diameter, and 70 ft. deep.

The shafts were reinforced vertically with wrought iron bolts, 1 in. in diameter, built in the brickwork and attached to wooden curbs at the bottom and top with nuts and screws. Iron hoops, 9 in. wide and ½ in. in thickness, were laid in the brickwork as the building progressed. The first shaft was built to a height of 42 ft. and then caused to sink by excavating the earth from the interior, using what is now commonly known as the open method of cassion construction, and the remaining 28 ft. of its height were added by underpinning. In spite of unequal settlement of the shaft as it was being sunk, no cracks developed in the masonry and, as a result, the second shaft was built to its entire height of 70 ft. before it was lowered. Richard Beamish, in his "Memoirs of the Life of Sir Marc Isambard Brunel" describes this construction and states that, after an unequal settlement of 7 in. on one side and 3½ in. on the other, "the surge was alarming, but so admirably was the structure bound together that no injury was sustained."

Brunel continued the use of reinforced masonry and in 1836 constructed test structures in an effort to determine the additional strength imparted to the masonry by the reinforcement. Other engineers became interested in this type of construction and in 1837 Colonel Pasley of the Corps of Royal Engineers conducted a series of tests on reinforced brick masonry beams and reported results comparable to those obtained by Brunel. Pasley's tests were planned to settle the then current argument as to whether or not the flat hoop iron really strengthened brick beams.

Three beams were built, each 18 in. wide, 12 in. (4 brick courses) deep, and all were tested on a 10-ft. span. One beam was built without reinforcement with the brick laid in neat cement. Brick in the second beam were also laid in neat cement but this beam was reinforced with 5 pieces of hoop iron; placed two in the top mortar joint, one in the middle joint and two in the bottom mortar joint; the latter of which obviously carried most of the tensile stress. The third beam was reinforced in the same manner as the second beam but the brick were laid in a mortar composed of one part lime and three parts sand.

The first beam failed at a load of 498 lb.; the second beam carried 4723 lb.; and the third beam failed at between 400 and 500 lb.; thus settling the dispute which had been current for some time.

As indicated by the placement of the reinforcement in Pasley's beams, the manner in which steel and masonry act together to resist forces was not understood at that time, and the empirical formulae derived from such tests could not be used to determine dimensions and reinforcement of structural members varying in cross-section or span from those tested. However, the interest in reinforced masonry construction continued and, with the increased production of portland cement for use in mortar, additional tests were conducted.

One such test which received widespread publicity was a reinforced brick beam tested at the Great Exposition in London in 1851. The "new cement", commercially known as "Portland Cement", was used in the construction. This test was highly successful, and the publicity which it received resulted in the more widespread use of portland cement in several European countries and to a lesser degree in the United States.

N. B. Corson published an article in the July 19, 1872, issue of "Engineering" (London) in which he reviewed the data obtained from the Exposition test beam, Brunel's test structures and tests of unreinforced masonry beams and arches, as well as the performance of a large number of masonry structures. From these data, Corson computed tensile stresses of unreinforced masonry and recommended an allowable tensile stress for use in the design of masonry lintels.

This appears to be the first recorded technical discussion of the relation of tensile strength of masonry to mortar strength. However, it does not recognize the full effect of the metal reinforcement in increasing the tensile strength of a masonry member.

During the 1880's builders conducted extensive experiments on reinforced concrete, and reinforced brick masonry structures were frequently cited to illustrate the strength which could be obtained through the combination of steel reinforcement and masonry. One such example was the Church of St. Jean de Montmarte, Paris, designed by M. A. deBoudot. The exterior brick walls are 4½ in. thick, reinforced with vertical wires through holes in the brick and horizontal wires in the mortar joints. One wall has an unsupported length of 29 ft. 6 in. for the full height of 115 ft. Another wall is approximately 38 ft. high with an unsupported length of 65 ft. The upper floor of this structure is supported by reinforced brick masonry columns 17⅜ in. square, spaced 39 ft. 6 in. on centers.

In the United States, Hugo Filippi, consulting engineer, now (1953) Vice President of the Illinois Brick Company, Chicago, Illinois, built and tested reinforced brick masonry beams in 1913. Later in 1919, L. J. Mensch, consulting engineer of Chicago, also tested reinforced brick beams in which the reinforcement was placed in a bed of mortar below the brick masonry. However, many of the data from these tests and others were never published and there was little if any exchange of information among those interested in the subject.

In 1923, the Public Works Department of the Government of India published Technical Paper No. 38 which is a comprehensive report by Under Secretary A. Brebner of extensive tests of reinforced brick masonry structures extending over a period of about two years. A total of 282 specimens was tested, including reinforced brick masonry slabs of varying thicknesses, reinforced brick beams, both reinforced and unreinforced columns, and reinforced brick arches. The tests reported by Brebner appear to be the first organized research on reinforced brick masonry and the data obtained from them provided answers to many questions which had formerly been raised regarding this type of construction. This research may, therefore, be considered as marking the initial stage of the modern development of reinforced brick masonry.

103. MODERN DEVELOPMENT

Following Brebner's report and his statement of a rational theory of design for reinforced brick masonry, its use increased, particularly in India and Japan. Both of these countries are subject to severe earthquakes and buildings which can be expected to withstand such shocks must be designed with relatively high resistance to lateral forces. Since structural steel and suitable form lumber are relatively expensive in these countries, engineers turned to reinforced brick masonry and adopted it as standard construction for public and important private buildings, as well as for many types of engineering structures, such as retaining walls, bridges, storage bins and chimneys.

Brebner writes: "In all, nearly 3,000,000 sq. ft. (of reinforced brick masonry) have been laid in the last three years" (prior to 1922), and Skigeyuki Kanamori, Civil Engineer, Department of Home Affairs, Imperial Japanese Government, is reported in the July 15, 1930 issue of Brick and Clay Record as stating: "There is no question that reinforced brickwork should be used instead of brickwork when any tensile stress would be incurred in the structure. We can make them more safe and stronger, saving much cost. Further, I have found that reinforced brickwork is more convenient and economical in building than reinforced concrete and, what is still more important, there is always a very appreciable saving in time." Structures described by Kanamori include sea walls, culverts and railway retaining walls, as well as buildings.

Research in the United States, sponsored by the Brick Manufacturers Association of America and continued by the Structural Clay Products Institute, has contributed much valuable material to the literature on reinforced brick masonry. Since 1924 numerous field and laboratory tests have been made on reinforced brick beams, slabs and columns and on full size structures. Some of the data obtained from these tests are summarized in Chapter 2.

In 1932 the National Brick Manufacturers Research Foundation enlisted the advice of a consulting committee formed of leading authorities on structural materials. This committee was known as the Reinforced Brick Masonry Research Board, and the following initial membership is listed in Report No. 6, February 1, 1932, of the Committee on Reinforced Brick Masonry:

Theodore Crane, *Professor of Architectural Engineering, Yale University, New Haven, Connecticut.*

Raymond E. Davis, *Professor of Civil Engineering, University of California, Berkeley, California.*

Herbert J. Gilkey, *Head, Department of Theoretical and Applied Mechanics, Iowa State College, Ames, Iowa.*

S. C. Hollister, *Professor of Structural Engineering, Purdue University, LaFayette, Indiana.*

Thomas R. Lawson, *Professor of Civil Engineering, Rensselaer Polytechnic Institute, Troy, New York.*

Inge Lyse, *Research Assistant Professor of Engineering Materials, Lehigh University, Bethlehem, Pennsylvania.*

F. M. McCullough, *Head, Department of Civil Engineering, Carnegie Institute of Technology, Pittsburgh, Pennsylvania.*

Charles E. O'Rourke, *Assistant Professor of Structural Engineering, Cornell University, Ithaca, New York.*

D. E. Parsons, *Chief, Masonry Construction Section, National Bureau of Standards, Washington, D. C.*

F. E. Richart, *Professor of Structural Engineering, University of Illinois, Urbana, Illinois.*

J. R. Shank, *Professor of Civil Engineering and Research Professor, Engineering Experiment Station, Ohio State University, Columbus, Ohio.*

Lorenz S. Straub, *Professor of Structural Engineering, University of Minnesota, Minneapolis, Minnesota.*

J. T. Thompson, *Professor of Structural Engineering, Johns Hopkins University, Baltimore, Maryland.*

R. G. Tyler, *Dean, College of Engineering, University of Washington, Seattle, Washington.*

Walter C. Voss, *Professor of Building Construction, Massachusetts Institute of Technology, Cambridge, Massachusetts.*

Alfred H. White, *Professor of Chemical Engineering, University of Michigan, Ann Arbor, Michigan.*

M. O. Withey, *Professor of Mechanics, University of Wisconsin, Madison, Wisconsin.*

This Board acted as a consulting body to the engineering staffs of the Brick Manufacturers Association of America and its successor, the Structural Clay Products Institute, in developing research programs, outlining test procedures and interpreting results.

In 1941 the Board enlarged its scope of activities to include research on all structural clay products and adopted the name: Structural Clay Products Research Board. Members at that time consisted of the following:

D. E. Parsons, *Chief, Masonry Section, National Bureau of Standards, Washington, D. C., Chairman.*

E. F. Wanner, *Structural Clay Products Institute, Washington, D. C., Secretary.*

George A. Bole, *Research Professor of Ceramic Engineering, Engineering Experiment Station, Ohio State University, Columbus, Ohio.*

E. A. Dockstader, *Chief Structural Engineer, Stone and Webster Engineering Corp., Boston, Mass.*

C. M. Dodd, *Professor of Ceramic Engineering, Iowa State College, Ames, Iowa.*

Paul E. Jeffers, *Consulting Structural Engineer, Los Angeles, California.*

Harry C. Plummer, *Director of Engineering and Research, Structural Clay Products Institute, Washington, D. C.*

F. E. Richart, *Research Professor of Engineering Materials, University of Illinois, Urbana, Illinois.*

William A. Rose, *Assistant Professor of Structural Engineering, New York University, New York City.*

John W. Whittemore, *Professor of Ceramic Engineering, Virginia Polytechnic Institute, Blacksburg, Virginia.*

M. O. Withey, *Professor of Mechanics, University of Wisconsin, Madison, Wisconsin.*

Henry Wright, *Associate Editor, Architectural Forum, New York City.*

In addition to its active participation in the initiation, correlation and supervision of research, this Board developed the first drafts of Recommended Practice and Standard Specifications for Reinforced Brick Masonry which have influenced largely the Building Code Requirements, Specifications and Construction Methods for reinforced brick masonry recommended in Chapters 3, 7 and 8, respectively.

In 1950 the Structural Clay Products Research Foundation was organized by the brick and tile manufacturers of the United States and Canada, and since that time the research on reinforced brick masonry, formerly initiated by the Structural Clay Products Research Board, has been included in the Foundation's program.

During the period from 1922 to 1950, research was conducted on both reinforced and unreinforced brick masonry at the National Bureau of Standards and at practically all of the principal engineering colleges of the United States.

As new data were developed through research, the erratic performance of some of the earlier reinforced brick test specimens could be explained and, one by one, the principal variables affecting the strength of reinforced brick masonry have been identified and, to a large degree, evaluated.

The suction of brick, defined as the weight of water that 30 sq. in. of the brick surface in contact with the mortar will absorb in 1 min., has come to be recognized as one of the most important factors affecting the strength of bond between mortar and brick. Research has also shown that "high flow" mortars; that is, mortars containing the maximum amount of water consistent with workability; have higher bond strength than low flow mortars.

The relation of brick unit strength to masonry strength has been studied and evaluated, also the relation of the shape of a masonry test specimen, particularly the ratio of its height to minimum thickness, to masonry strength has been determined.

Numerous investigations, as well as the construction of many reinforced brick masonry buildings, have demonstrated the superiority of grouting over the conventional methods of bricklaying.

Effective methods of locating reinforcement, so that the masons can work with the least hindrance, have been developed and maximum permissible spacing of rods determined.

At the present time (1953) the American Standards Association's Sectional Committee A41 on Building Code Requirements and Good Practice Recommendations for Masonry is developing proposed American Standard Building Code Requirements for Reinforced Brick Masonry, and it is anticipated that this standard will shortly be approved. (See Chapter 3.)

While, as indicated in Section 101, reinforced brick masonry is relatively new as compared to the ancient masonry craft, it is by no means untried. During the past 30 years, it has definitely progressed to the point where engineers may now design in reinforced brick masonry on a rational basis, and, based on the materials used in the construction, can predict strengths from which to fix working stresses with reasonable accuracy.

104. ADAPTABILITY

As previously indicated, reinforced brick masonry has been used for the construction of a wide variety of structures and, in those countries where labor costs are very low, one of its principal uses has been for the construction of floor and roof slabs. However, in the United States, its most extensive use has been in the construction of vertical members, such as walls and columns. Since no forms are required for these members, reinforced brick masonry is competitive with reinforced concrete and walls of minimum thickness and light structural members can be constructed at substantially less cost in reinforced brick masonry than in reinforced concrete.

In 1933 the Brick Manufacturers Association of America published "Brick Engineering, Vol. 3, Reinforced Brick Masonry", by Hugo Filippi. Regarding the uses of reinforced brick masonry, the author states: "Reinforced brick masonry is well adapted for use in the following types of structures, either wholly or in part: Buildings, Culverts and Bridges; Retaining Walls and Dams; Reservoirs; Sewers and Conduits; Tanks and Storage Bins; Chimneys and Circular Constructions; Abutments, Piers, Trestle Bents, etc.

"In the United States alone, during the past year and one-half, more than 40 individual jobs of reinforced brick masonry have been built, consisting of such distinctive types of construction as highway bridges, storage bins, industry track trestle piers, floor and roof slabs, beams, girders and long lintels. At the present time approximately 50 additional jobs are either under construction or under consideration in various parts of the country."

During the period referred to by Filippi, the development and use of reinforced brick masonry in the United States were in their early stages. Since that time thousands of such structures have been built and reinforced brick masonry construction has been adopted as standard practice for various types of structures in many areas.

Norman W. Kelch, Technical Director of the Associated Brick Manufacturers of Southern California, writes in August 1951 in a report to the Veterans Administration on the use of reinforced brick masonry in Southern California: "The most eminent architects and structural engineers have been designing in reinforced brick masonry for many years past. The buildings include:

"Large Industrial and Commercial: (a) Sears' new warehouse, 206 ft. by 989 ft. by 24 ft. high bearing walls; (b) Two- and three-story stores; (c) Many banks; 4-story office buildings in Beverly Hills; (d) Many industrials smaller in area than Sears.

"Churches: Small and very large, all bearing walls, some with reinforced brick masonry columns and girders.

"Public and Parochial Schools: Bearing walls, columns and girders with floors and roofs of reinforced concrete; including dormitories of colleges—Whittier College, University of Southern California, University of California-Los Angeles.

"Public Buildings: Playground buildings, branch public office buildings, county and city public buildings, also many branch post offices.

"State Buildings: (Now under construction) (a) Boys' School, San Luis Obispo, Adam Arras & Son, San Francisco—contractors, about $2,000,000; (b) Armory, Long Beach; (c) State Institute for Women, Corona, McKee Construction Company—contractors, about $3,500,000; Paul E. Jeffers, eminent structural engineer, designed the structural work on the foregoing buildings; (d) Employment Bureau, Compton. Many other state buildings have been completed."

With this report Kelch includes a list of 43 school buildings built in the Los Angeles area which he designates as "a few" of the school buildings constructed in that area and which are representative of "the constantly increasing use of reinforced brick masonry for elementary and junior high schools and colleges."

Report No. 5 of the Committee on Reinforced Brick Masonry of the National Brick Manufacturers Research Foundation, dated February 1, 1932, describes "all of the known structures, ancient and modern", built of reinforced brick masonry, beginning with the shafts designed by Brunel and constructed in 1825. The report lists 15 structures in addition to those constructed in India and Japan, reported by Brebner and Kanamori, respectively, and states: "No doubt much other work has been built of this construction, but a careful inspection of available literature fails to reveal more than set forth here."

A similar list of structures erected of reinforced brick masonry since the 1932 compilation of the committee would require a volume much larger than this one if the location and information regarding such structures could be obtained. No attempt will be made to present such a listing.

The structures described in the following pages do not represent all of the uses to which reinforced brick masonry is adaptable; however, they are selected to show the versatility of this type of construction and to indicate its widespread acceptance by engineers and architects during the past 20 years.

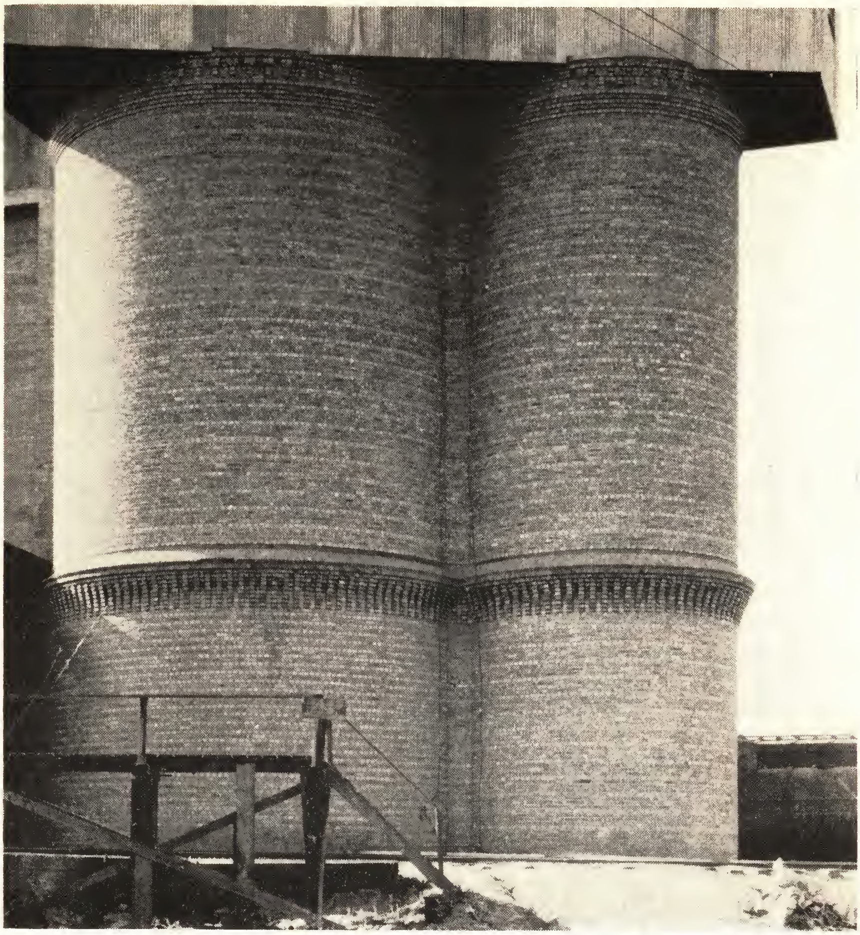


Fig. 1-1

Reinforced Brick Masonry Sand Storage Bins, Wedron, Illinois

Storage Bins. In 1931 the Wedron Silica Company of Wedron, Illinois, constructed two reinforced brick masonry sand storage bins, 25 ft. and 16 ft. in diameter and both 52 ft. high. These bins, shown in Fig. 1-1, were designed by Hugo Filippi and are described in the May 19, 1931 issue of Brick and Clay Record.

Fig. 1-2 is the plan of these structures, and the wall sections shown in Fig. 1-3 indicate the amount and placing of the reinforcement.

Reinforced masonry is particularly well adapted for the construction of circular storage bins, and for such structures the principles of prestressing may be effectively employed. Since the construction of the Wedron sand bins, hundreds of similar structures have been built for the storage of silage, grain, coal and other materials.

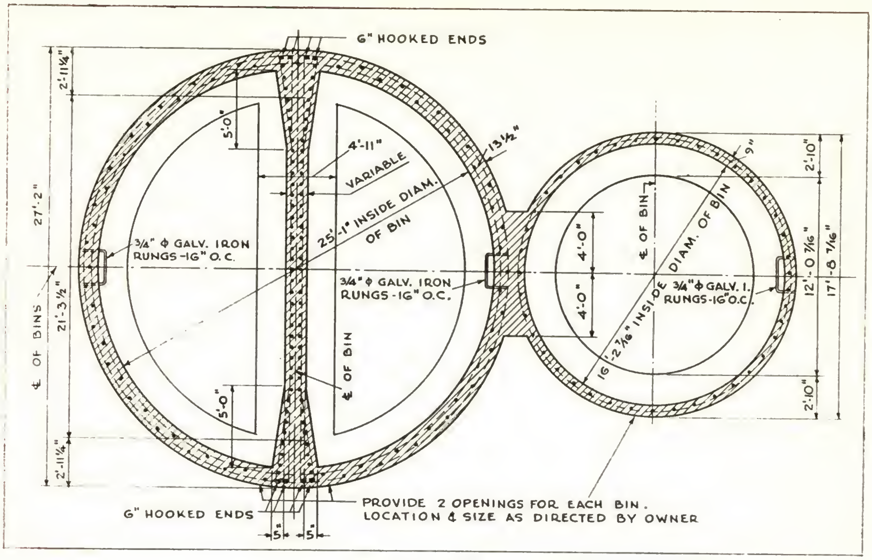
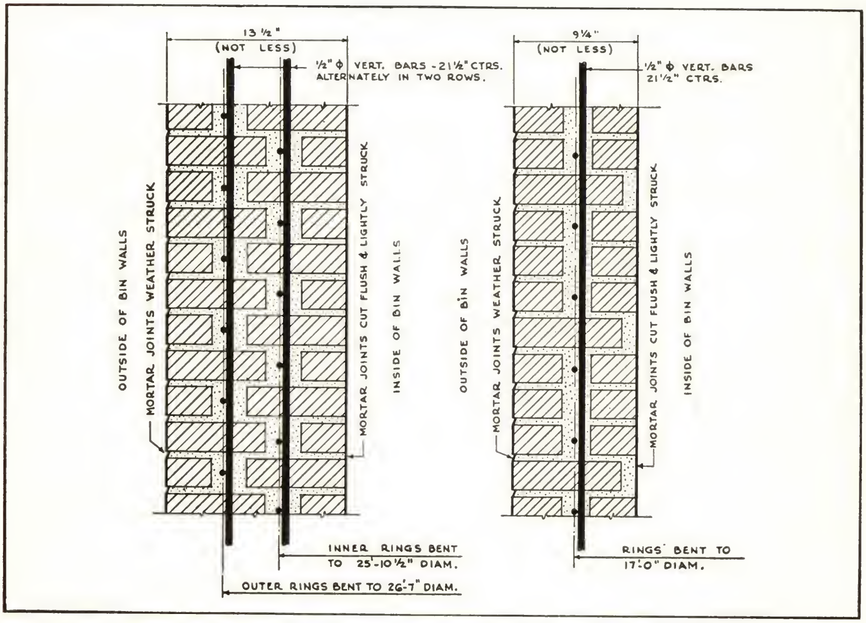


Fig. 1-2

Plan of Reinforced Brick Masonry Sand Storage Bins



Note: Brick headers such as used in these walls should not be used in recommended grouted construction. Size and spacing of horizontal rings vary with height of bin.

Fig. 1-3

Typical Reinforced Brick Masonry Bin Walls

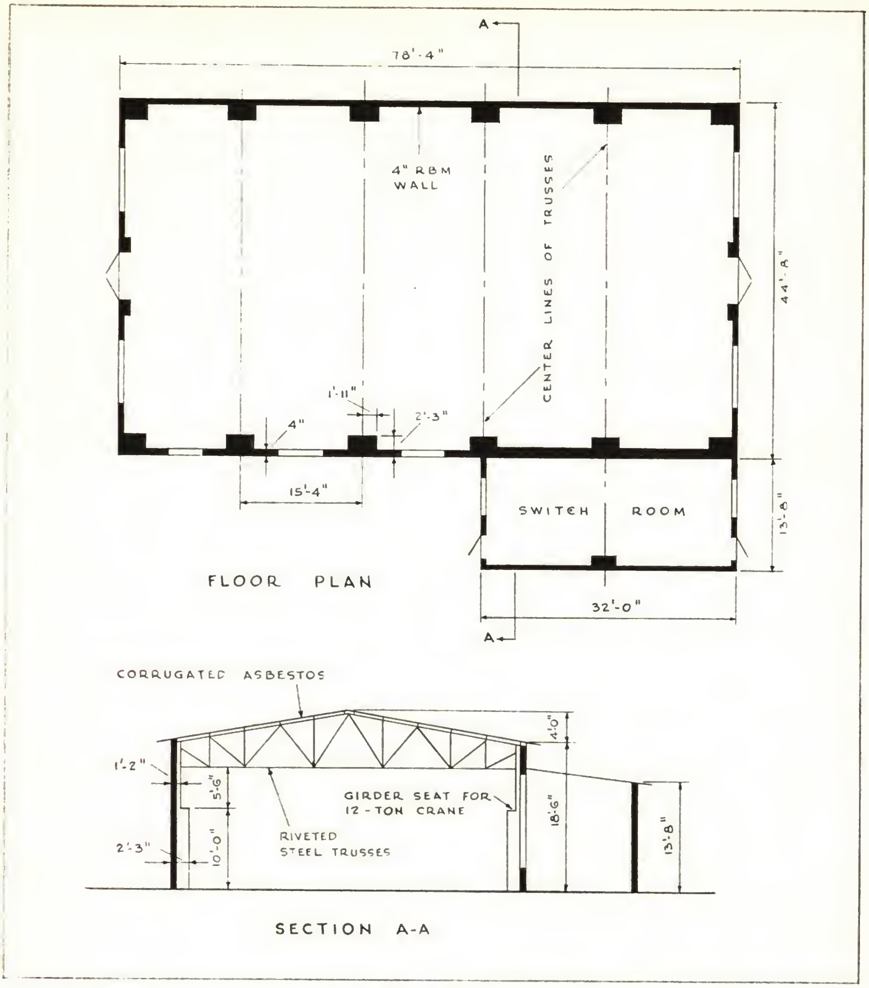


Fig. 1-4

Plan and Section of Compressor House, Wood River, Illinois

Panel Walls. In 1932 the Standard Oil Company of Indiana constructed a "compressor house" of reinforced brick masonry at its refinery at Wood River, Illinois. This construction is described in the April 1933 issue of Civil Engineering by H. S. Haworth, Structural Engineer, Standard Oil Company of Indiana.

Fig. 1-4 is a plan and section of the compressor house and gives the principal dimensions of the structure. The side columns, 23 in. by 27 in. up to the crane girder seat, are designed to support a 12-ton crane and are reinforced with six ¾-in. round bars with ¼-in. round ties spaced approximately 9½ in. on center. The 4-in. panel walls are reinforced with ⅜-in. round rods spaced approximately 9¾-in. center to center, with additional ⅜-in. rods placed ½-in. from the outside face of the wall and extending from quarter span to quarter span continuously past the columns.

The author describes the construction of this building in considerable detail and concludes: "It appears to me, after my experience with this building, that the use of reinforced brick masonry can be expected to become quite general. This is especially true of industrial work because it is economical to construct; it presents a more pleasing appearance than corrugated iron or corrugated asbestos; the heat loss through a 4-in. brick wall is about 60 per cent of that through corrugated asbestos; and its maintenance will probably be small. It also appears that reinforced brick masonry may be expected to supplant reinforced concrete in many places where there are large vertical surfaces, because it obviates the necessity of providing forms for such surfaces."

Haworth's prediction that "the use of reinforced brick masonry can be expected to become quite general" has been fully realized, and reinforced brick masonry panel walls, as well as bearing walls, have been used extensively in those sections of the United States where building regulations require that structures be designed to resist high lateral forces resulting from earthquake, tornado or other causes.

Present trends, however, indicate a tendency on the part of all organizations concerned with building codes and design criteria to emphasize requirements for the resistance of buildings to lateral forces and, as a result, the use of reinforced masonry panel wall construction may be expected to increase.

Highway Bridges. In 1934 the Ohio Department of Highways constructed a highway bridge on State Route 39, just east of Sugar Creek, Ohio, in which reinforced brick masonry was used extensively.

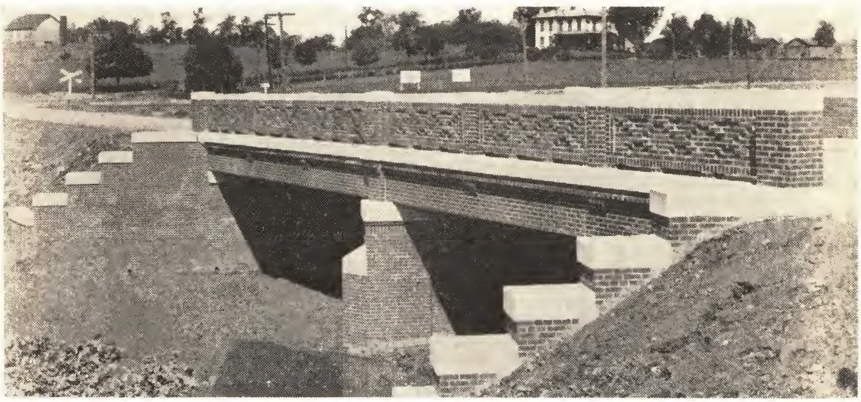


Fig. 1-5
Reinforced Brick Masonry Highway Bridge, Sugar Creek, Ohio

Fig. 1-5 is a photograph of this bridge and its construction is described in the February 28, 1935 issue of Engineering News-Record by J. R. Burkey, Chief Engineer of Bridges of the Ohio Department of Highways. The abutment walls and pier are of reinforced brick masonry as well as the hand-rail, and Burkey reports that the brick portions of the bridge were built in a shorter time than if concrete had been used, and that the cost compares favorably with comparable types of construction.

Reinforced brick masonry has been used extensively for retaining walls, culverts and bridges during the past 20 years and it is particularly well adapted for use in parks and other areas where consideration is given to architectural treatment.

Fig. 1-6
Reinforced Brick Masonry Bleachers, Maple Heights, Ohio

Bleachers. In 1936 the Board of Education of Maple Heights, a suburb of Cleveland, Ohio, completed the construction of an athletic field as a WPA project. This project included the construction of reinforced brick masonry bleachers and a reinforced brick masonry retaining wall, and is described in the August 27, 1936 issue of Engineering News-Record by Harry C. Plummer, Consulting Engineer on the project.

Fig. 1-6 is a photograph of the risers. The bleachers are approximately 300 ft. long and 15 ft. deep. The riser beams are spaced 5.2 ft. center to center, have an effective depth of 8½ in. and are reinforced with four ½-in. square bars. Shear reinforcement consists of ⅜-in. and ¼-in. stirrups.

The author states: "In estimating the project, it was found that the cost of materials, both for the bleachers and for the reinforced brick retaining wall, was less using reinforced brick masonry than reinforced concrete, due to the elimination of form lumber. In addition to the saving in material cost, the architecture harmonized with that of the existing school buildings."

This project illustrates the adaptability of reinforced brick masonry for structures where it is desired that their appearance harmonize with existing buildings. Prior to the use of reinforced brick masonry, this effect was often achieved by veneering reinforced concrete structures with brick at a substantial increase in cost. Through the use of reinforced brick masonry, both architectural treatment and structural integrity can be obtained at low cost.

School Building. In 1937 the Los Angeles County Board of Education constructed the Vermont Avenue School of reinforced brick masonry. Charles A. Fork was consulting engineer, and in describing this structure in the August 1937 issue of Building Standards Monthly, he states: "The new Vermont

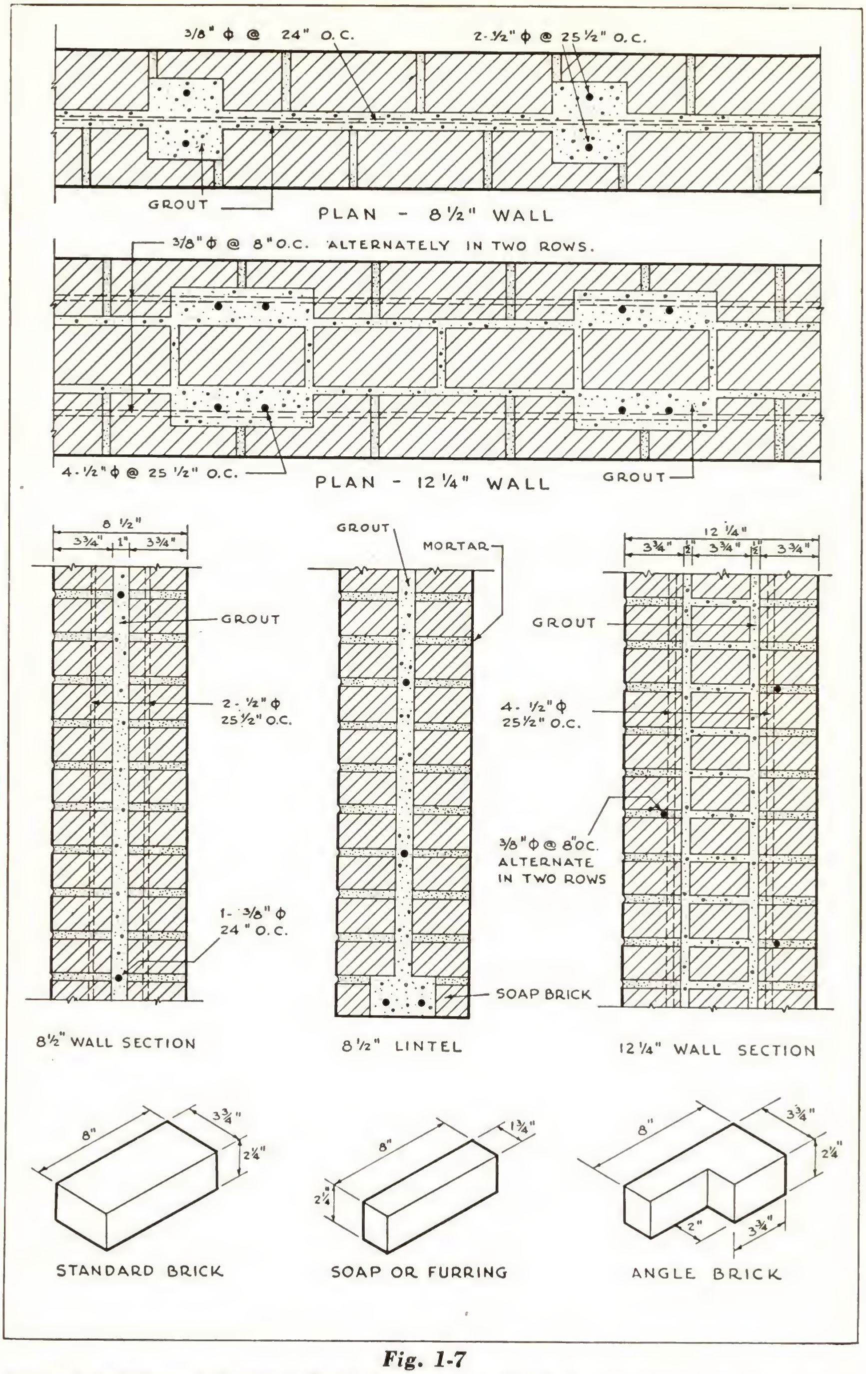


Fig. 1-7

Shaped Brick and Typical Wall Sections as Used in the Construction of the Los Angeles Vermont Avenue School

Avenue School in Los Angeles, California, is an outstanding example of what can be done to make brick construction safe in an area subject to earthquake shocks.

"The east and west wings of the school are two stories high, connected in front with a one-story unit. The horizontal floor area is 38,700 sq. ft. Total cost of the building proper was approximately $180,000 or $4.62 per sq. ft. of floor area. The school cost less than a similar job of concrete. This is significant in view of the fact that face brick were used to face the 13-in. exterior walls and interior corridors. Unusual features in this school are the use of shaped brick and the grouting method for filling the interior joints."

Fig. 1-7 shows the "shaped" brick and typical wall sections as used in the construction of the Los Angeles Vermont Avenue School.

Housing Project. In 1940 the Los Angeles Housing Authority constructed the Ramona Gardens housing project of reinforced brick masonry. This project, shown in Fig. 1-8, consists of 112 two-story buildings with 8-in. thick reinforced walls for the first story and 7-in. thick reinforced walls for the second story.

Fig. 1-9 shows units and wall sections of "Groutlock" construction developed by the Simons Brick Company and used in the Ramona Gardens project.

Fig. 1-8

Ramona Gardens Housing Project, Los Angeles, California

As indicated by the wall sections, the V or soffit brick provide space for the reinforcement at the bottom of a lintel or beam and the beveled edges of the stretcher unit, in addition to forming a grout key, provide additional space for the horizontal steel. Various other shapes designed to accomplish the same purposes are also available.

The cost of the Ramona Gardens project was $2,004,000.00 and alternate bids were taken for both reinforced brick masonry and reinforced concrete construction. Reinforced brick masonry construction was bid low by all of the five contractors who submitted bids. The low bidder, Baruch Corporation, to whom the contract was awarded, bid reinforced brick masonry approximately $35,000.00 under reinforced concrete.

Rural Housing. In 1941 Robert B. McKnight, District Engineer for the Louisiana-Mississippi Brick Manufacturers Association, utilized reinforced brick masonry in the design of three-bedroom masonry homes for rural housing.

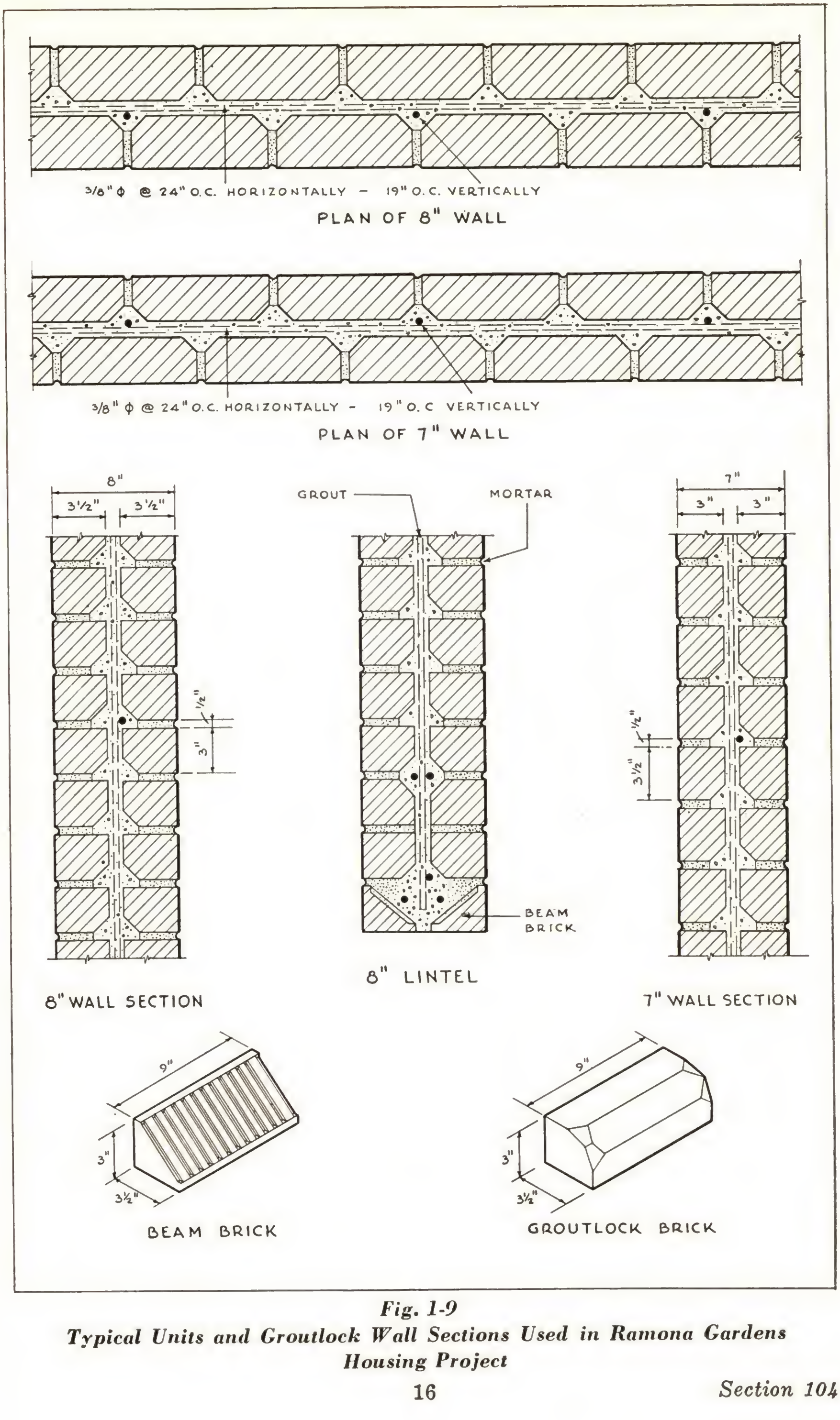


Fig. 1-9
Typical Units and Groutlock Wall Sections Used in Ramona Gardens Housing Project

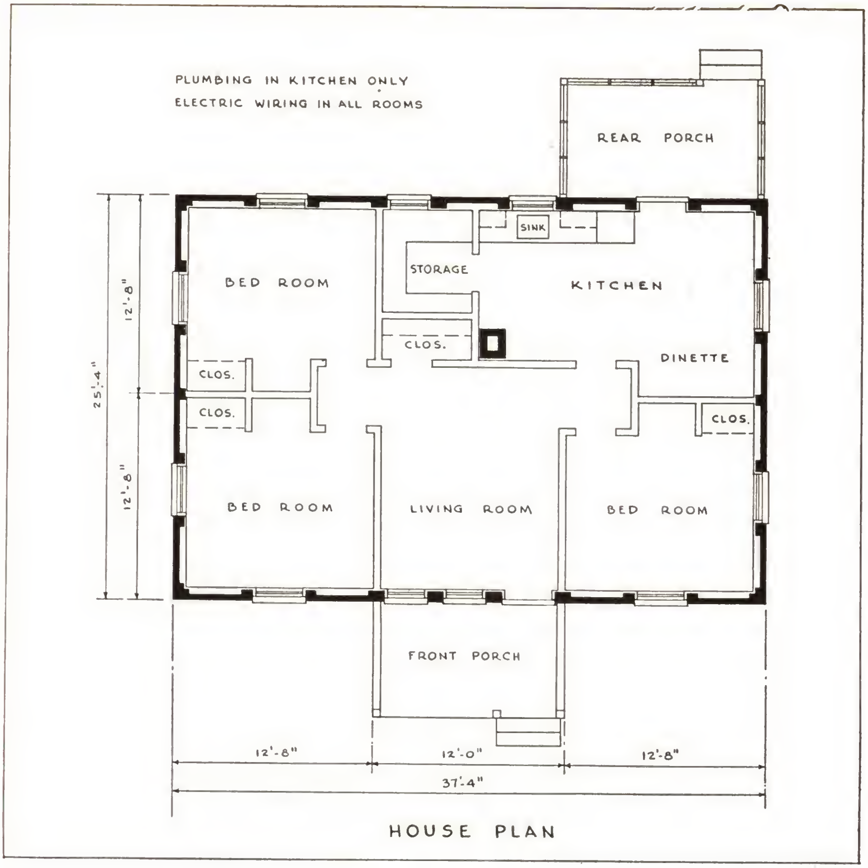


Fig. 1-10

Typical Floor Plan of Homes Constructed by the Lee County Housing Authority, Tupelo, Mississippi

Fig. 1-10 is a typical floor plan of these houses, a large number of which were constructed by the Lee County Housing Authority, Tupelo, Mississippi. The wall construction is a 4-in. brick wall, known as the "Economy" wall and is illustrated in Fig. 1-11. The foundation consists of 8-in. by 16-in. brick piers spaced approximately 5 ft. 8 in. center to center, with a reinforced brick grade beam approximately 18 in. deep supporting the structure.

McKnight reports that the successful contractor's bid on these houses in 1941 was $1,538.30 which was "lower than competing bids for frame construction."

In many areas this type of construction can be used most effectively for low cost housing or other light structures and its low maintenance cost as compared to frame construction, particularly in southern climates, makes it highly competitive with frame when costs are figured for a 10- to 20-year period.

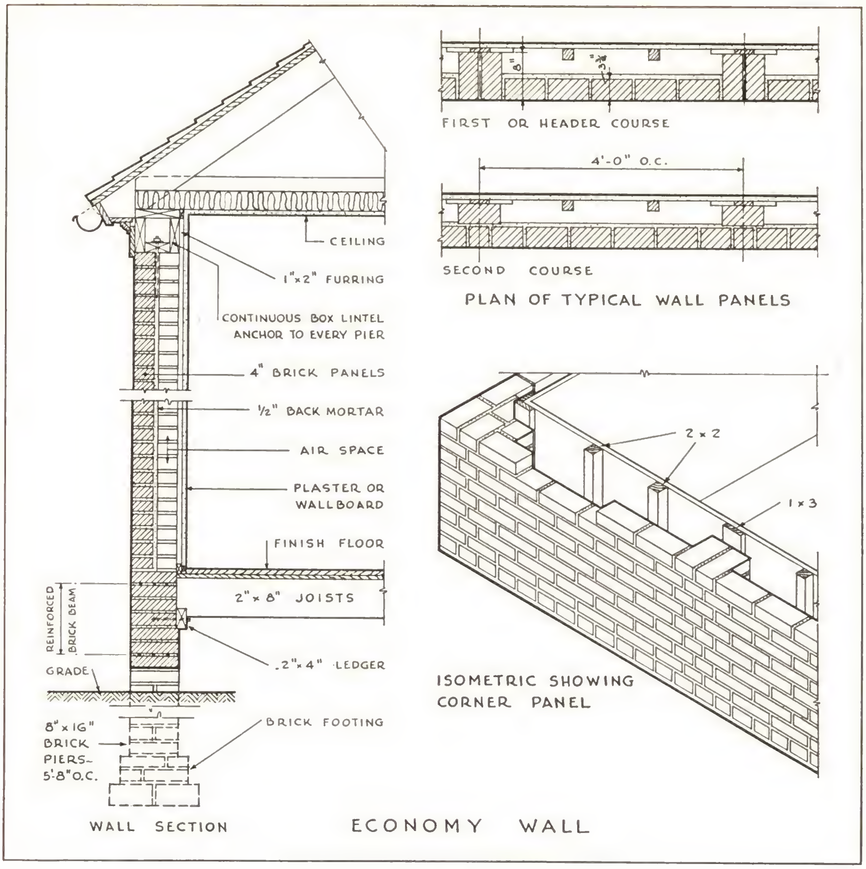


Fig. 1-11

Typical Economy Wall Construction Used in Lee County, Mississippi, Homes

Lintels and Beams. During 1941 and 1942 Camp Lejeune, the Marine base at New River, North Carolina, was constructed. Design and construction were under the supervision of the Bureau of Yards and Docks, Department of the Navy, and originally the lintels were specified to be steel angles. Following Pearl Harbor, the size of the project was substantially increased and at the same time steel became critical. As a result, the project was redesigned and reinforced brick masonry was used for the more than 6,000 lintels and beams, ranging in span from 3 ft. to over 12 ft.

This project is described in the November 1943 issue of Brick and Clay Record which states: "Savings in steel, made possible by using reinforced masonry lintels, were significant. On the standard warehouse the original design which used standard steel angles as lintels required 11,682 lb. of steel for lintels alone. A redesign of the same building using reinforced lintels required but 1,126 lb. of reinforcing steel for lintels, a net saving of 10,556 lb. of steel."

The use of reinforced brick lintels has now become standard practice in many architectural and engineering offices, due not only to the fact that the cost is considerably less than for steel angles but, more importantly, because the danger of steel angles corroding and causing cracks in the masonry is eliminated. In addition, the masonry is more resistant to rain penetration if reinforced brick lintels are used, since the mortar joint which contains the horizontal leg of the lintel angle is always a vulnerable spot for rain penetration.

Locomotive Repair Shop. In 1943 the Union Railroad Company of East Pittsburgh, Pennsylvania, constructed a locomotive repair shop approximately 185 ft. long by 53 ft. 8 in. wide in which reinforced brick pilasters, spaced 14 ft. center to center, were used to support a 19-ton crane.

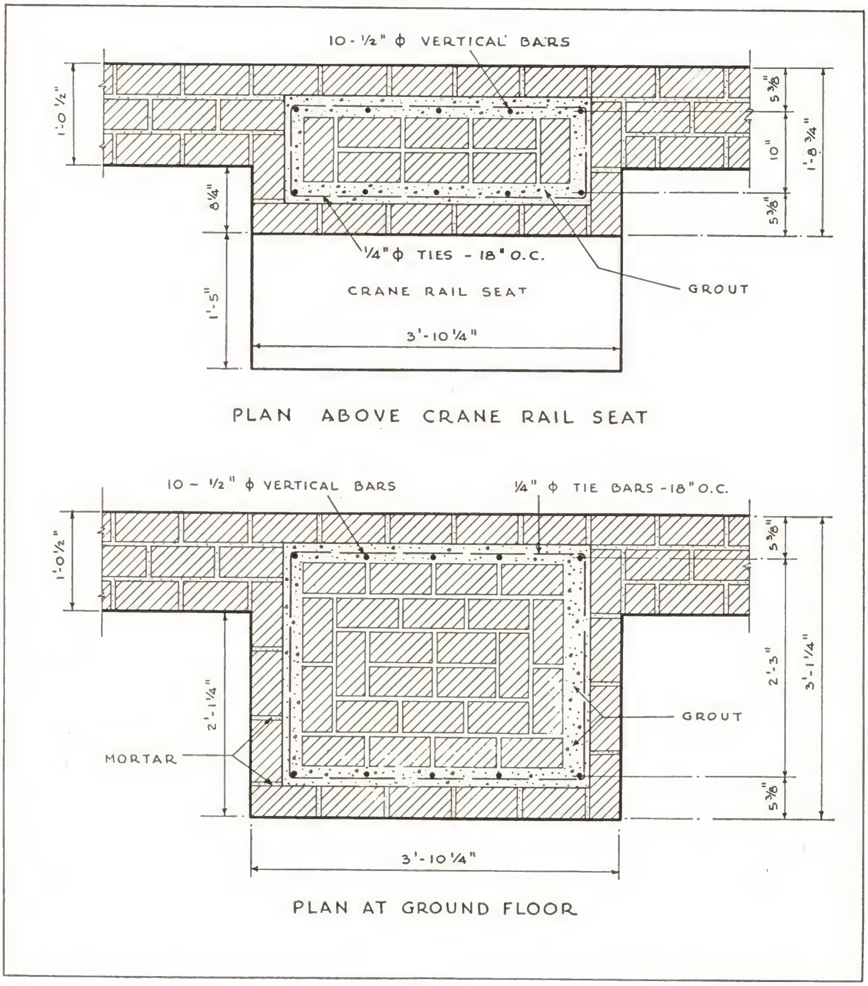


Fig. 1-12
Cross-section of Pilasters of Locomotive Repair Shop for Union Railroad Company of East Pittsburgh, Pennsylvania

Fig. 1-12 is a cross-section of these pilasters showing the amount and placement of the reinforcement.

The Union Railroad Company's engineers adopted this type of construction, both as a means of eliminating the use of structural steel which was critical at the time and also as a means of reducing the cost of the completed structure.

Reinforced brick masonry is particularly well adapted for walls or columns designed to support heavy loads and, since the structural member becomes a part of the enclosing walls, substantial economies can be effected.

Multi-Story Apartments. In 1947 the architectural firm of Holsman, Holsman, Klekamp and Taylor, Chicago, Illinois, designed and constructed multi-story apartment buildings, five stories high, with reinforced brick load-bearing walls. Subsequent projects were built nine stories high.

The construction developed by this firm is known as "Rowlock Reinforced Brick Masonry", since all brick are laid on edge (rowlock). Fig. 1-13 shows the details of this construction for both 8-in. and 10-in. walls.

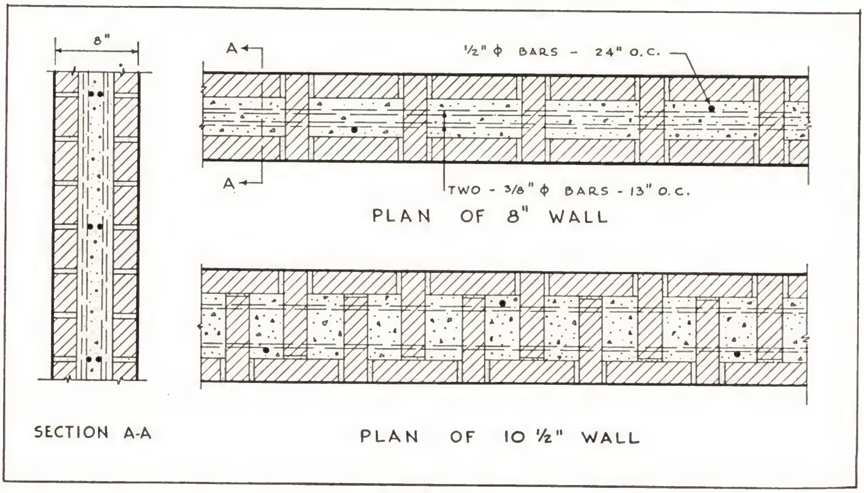


Fig. 1-13
Details of Rowlock Reinforced Brick Masonry Wall Construction

Perhaps the most dramatic use of this type of construction is in the Lunt-Lake apartment buildings in Chicago which are nine stories high and were completed in 1950. Pace Associates, Holsman, Holsman, Klekamp and Taylor, and Frank J. Kornacker were associated architects and engineers on this project. Walls of the first story are 10½ in. thick and the walls of the remaining eight stories are 8 in. thick.

As compared to unreinforced masonry structures, this appears to be a daring design, utilizing the compressive strength of the masonry to a much greater degree than is common, and for such a design scientific methods of controlling materials and adequate inspection should be employed at all times.

Building regulations in the Chicago area do not require that buildings be designed to resist lateral forces due to earthquakes, and for this reason the walls of the Lunt-Lake apartments seem very light for the number of stories when compared to the walls of reinforced brick buildings constructed on the West Coast where seismic lateral force requirements apply.

Rowlock reinforced brick masonry was used for the construction of an apartment dormitory, shown in Fig. 1-14, which is located at the State Sanitorium, McCain, North Carolina. This building was designed by Architect F. Carter Williams of Raleigh, N. C., and built by the Wilson-Ledford Construction Company at a cost of $50,000.00 exclusive of mechanical equipment.

Fig. 1-14

Apartment Dormitory, State Sanitorium, McCain, North Carolina, Constructed of Rowlock Reinforced Brick Masonry

Fig. 1-15

Typical All-Rowlock Bond Pattern as Used in Apartment Dormitory

Fig. 1-15 shows the typical all-rowlock bond pattern and the vertical reinforcement which consisted of ½-in. bars on 13-in. centers. Horizontal reinforcement of ⅜ in. bars on 16-in. centers was placed as the walls were built.

As of this date (1953) rowlock reinforced brick masonry construction has not been used extensively by others than the developers due, perhaps, to the unfamiliarity of both engineers and contractors with this type of construction. However, the originators of the system state that its use results in substantial cost reduction as compared with skeleton frame construction, and Henry K. Holsman writes in the June 1949 issue of Brick and Clay Record:

"If brick masons and manufacturers will adopt and promote this type of reinforced brick construction, it will put the mason and the brick industry on an equal competitive footing with other methods of building construction, if not far in advance of them. This idea should be quite interesting to apprentices and engineering students since the economies derived from reinforced brickwork will induce more brick building to be built. In my judgment it can be developed so as to make the concrete skeleton method of construction quite obsolete. To abolish the temporary forming in building construction, which sometimes and quite often costs as much as or more than the structure itself, will be a great economical boon to the general public who, after all, must pay the extra cost of any inefficient or unnecessary methods of building construction."

Power Plant. In 1951 the Pacific Gas and Electric Company constructed a 300,000-kw steam generating plant of reinforced brick masonry at Antioch, California, about 40 mi. east of San Francisco. Fig. 1-16 is a photograph of this plant which was designed and built by the Bechtel Corp. Its construction

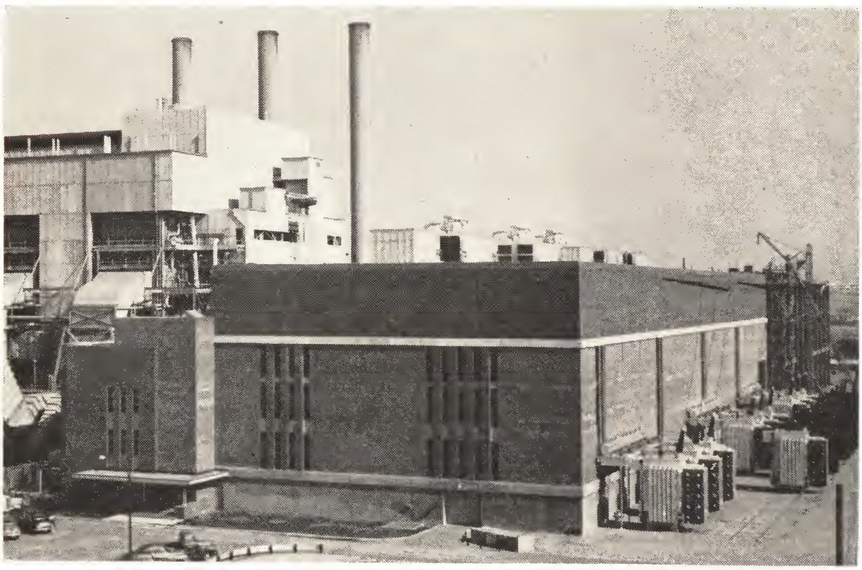


Fig. 1-16

Steam Generating Plant of Pacific Gas and Electric Company, Antioch, California, Constructed of Reinforced Brick Masonry

is described in the July 5, 1951 issue of Engineering News-Record by Walter L. Dickey, Supervising Structural Engineer, Power Division, Bechtel Corp. The following is excerpted from Dickey's article.

"The plant at Antioch is one of the largest—if not the largest—brick buildings ever designed to resist such a high earthquake loading. Another structural feature is use of reinforced masonry bond beams that span as much as 27 ft.

"PG & E engineers placed the lateral force coefficient for design purposes at 20 per cent of gravity. The high coefficient was selected for this structure because the plant is in an active earthquake zone and because the structure houses a vitally important facility. The same factor would have been used for design purposes no matter what material was specified for the walls of the building. It is significant to note that brick can be designed and built to resist such forces.

"In this instance, much of the lateral loading is resisted by the structural steel frame of the building. But the reinforced grouted masonry walls and bond beams must transmit to the columns such lateral load as is developed between columns.

"Reinforced, grouted brick masonry was selected for the 89-ft. high walls —after studies of several materials—as the most economical, unit type, masonry construction that would resist the lateral loads the walls were required to carry. Low maintenance, favorable installation cost, and pleasing architectural results were factors in the choice of reinforced brick.

"The architectural designers were able to achieve a pleasing effect on exterior walls by combining two surface textures of a commercial building brick, namely: rug-cut and smooth-face. These are supplemented for accent by precast concrete elements. Architectural treatment of the building required narrow 30-ft. high slot-type windows generally in groups of three. Mullions in this case were designed as reinforced brick columns the height of the window openings.

"Precast, hollow concrete units involving special exposed colored aggregate were used for an architectural band encircling the building. These units were designed as part of bond beams.

"Interior masonry partitions are lightweight concrete blocks with grout filled reinforced cells. Some glazed structural tile partitions are also used and these are reinforced by similar grouting and reinforcing systems."

VA Hospital. In August 1952 the Veterans Administration awarded a contract for the construction of a 1000-bed hospital in Los Angeles, California, to Gust K. Newberg at a total bid of $18,487,809.00. This contract includes the construction of eight 2-story and eleven 1-story buildings of reinforced brick masonry designed to conform to the local earthquake lateral force requirements. This is the first extensive use of reinforced brick masonry by the Veterans Administration and the bid on this project indicates substantial savings in the cost of reinforced brick masonry over comparable types of construction.

Reinforced Brick Pavements. While the scope of this book is concerned primarily with building construction, it seems appropriate in discussing the various uses of reinforced brick masonry to mention its adaptability for highway and street paving. Fig. 1-17 shows a cross-section and detail of the reinforced brick pavement developed by the Poston Brick and Concrete Products Company of Springfield, Illinois. The paving brick are 8 in. square and

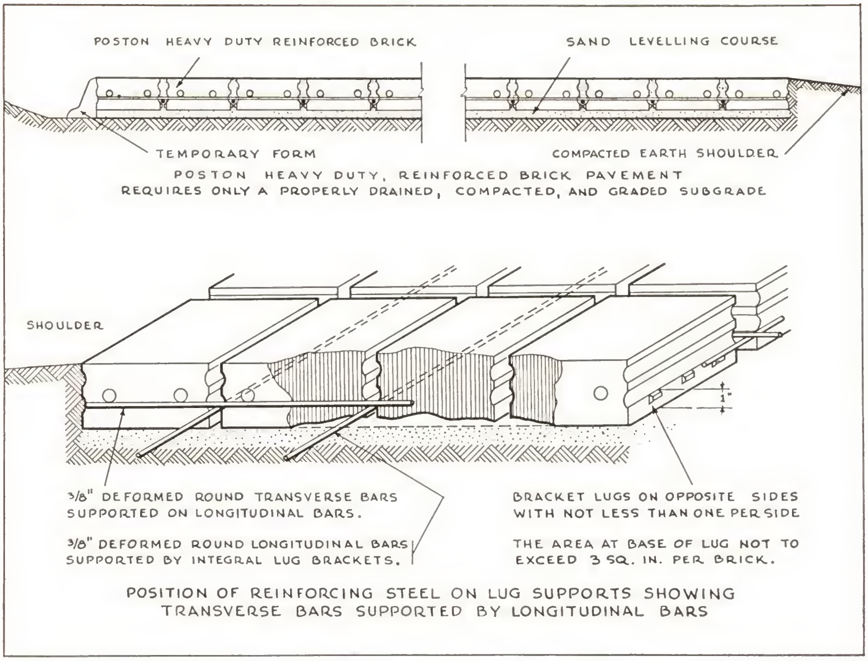


Fig. 1-17

Details of Postonway Reinforced Brick Pavements

4 in. in depth and, as indicated in Fig. 1-17, the pavement is reinforced longitudinally and transversely; the transverse rods being supported on the longitudinal reinforcement which is supported by lugs on the sides of the brick.

Regarding this construction, the Poston Company states: "The cost of building the reinforced brick pavement will vary with location and other conditions, but estimates, predicated upon typical conditions, indicate that it will not exceed, and probably not equal, that of standard concrete slab construction. This conclusion seems confirmed by a very careful check on costs, conducted by capable engineers, during the building of over a mile of this pavement by the State of Illinois in 1940."

105. COSTS

Costs of reinforced brick masonry, as of other types of construction, vary materially in different sections, depending upon supply and productivity of labor, on-site cost of materials, and the construction market. No attempt will be made to state absolute costs; however, the following estimates obtained from experienced contractors indicate the relative costs per square foot of wall area of reinforced brick masonry and reinforced concrete walls.

Prices per square foot of wall area from Los Angeles, California, contractors are given in Table 1-1 for 8-in. and 12-in. thick reinforced brick masonry and reinforced concrete walls with minimum reinforcement as required in the Uniform Code of the Pacific Coast Building Officials Conference.

TABLE 1-1

ESTIMATED COSTS PER SQUARE FOOT OF WALL AREA

Los Angeles, Calif., Contractors August 1951	Type	Nominal Thickness	
		8-in.	12-in.
A	RBM	$1.28	$1.68
	Concrete	1.65	1.90
B	RBM	1.53	2.15
	Concrete	1.85	2.15

This code provides that reinforced brick masonry walls shall be reinforced with "not less than 0.002 times the cross-sectional area of the wall" and that reinforced concrete walls "shall be reinforced with an area of steel in each direction, both vertical and horizontal, at least equal to 0.0025 times the cross-sectional area of the wall."

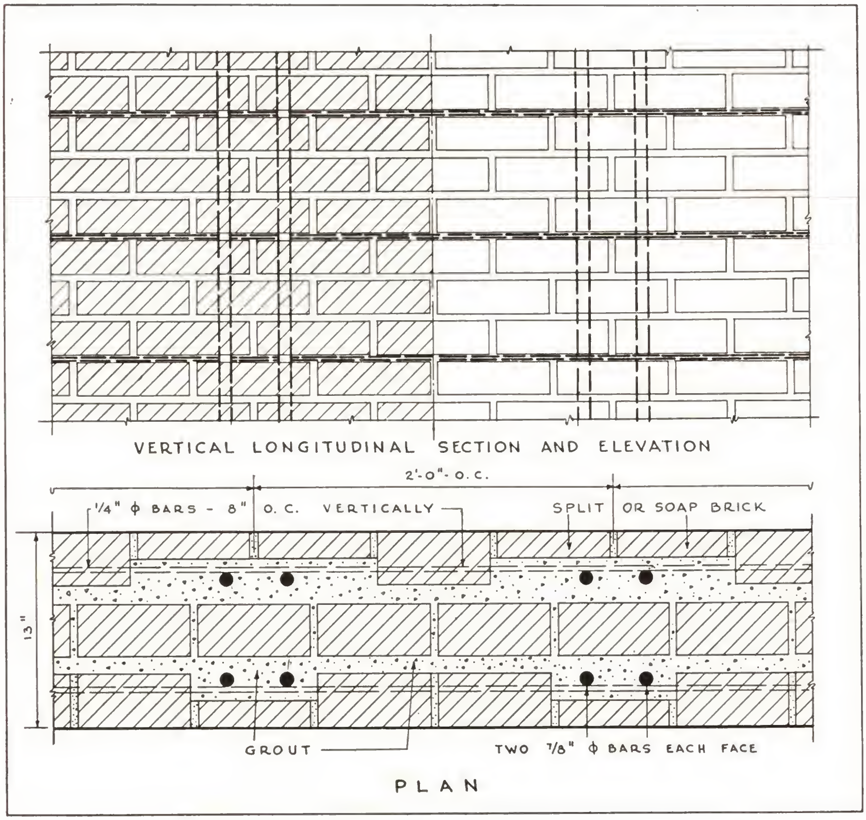


Fig. 1-18

Details of Reinforced Brick Wall Designed for Blast Resistance

Contractor A's prices include all labor and material, but no equipment, overhead or profit. Contractor B's prices include labor, materials, scaffolding and equipment items as well as compensation, but no profit.

Fig. 1-18 is a section of a reinforced brick masonry wall designed for blast resistance in accordance with criteria established by the Veterans Administration for blast resistant hospital construction. Table 1-2 shows estimated costs per square foot of this wall and quotations submitted by four Cleveland, Ohio, contractors.

TABLE 1-2

ESTIMATED COSTS PER SQUARE FOOT OF WALL AREA

	Estimate	Cleveland, Ohio, Contractors			
		A	B	C	D
Total November 1952 Materials Cost	$1.168	$1.141	$1.21	$1.21	$1.11
Total November 1952 Labor Cost	1.804	1.851	2.85	2.14	1.35
Sub-total Cost per sq. ft. of wall	2.972	2.992	4.06	3.35	2.46
Plus Profit and Overhead	.5944	.598	.81	.27	.39
Grand Total per sq. ft.	$3.57	$3.59	$4.87	$3.62	$2.85

For the area in question, Cleveland, Ohio, the costs in Table 1-2 are approximately the same as the cost of a reinforced concrete wall of equivalent strength.

In general, it will be found that reinforced brick masonry walls with minimum reinforcement can be constructed at from 10 to 20 per cent less cost than equivalent reinforced concrete walls, and that heavily reinforced brick masonry walls can be constructed at approximately the same cost as equivalent reinforced concrete walls.

CHAPTER 2

PROPERTIES OF REINFORCED BRICK MASONRY

201. INTRODUCTION

As with other types of construction, the performance and durability of reinforced brick masonry structures depend upon: (a) the quality of the materials—brick, mortar, grout and reinforcement—of which the construction is composed; (b) the workmanship and methods of construction; (c) the design.

In considering mortar and grout which are bonding agents and are relied upon to produce a homogeneous construction, it will be found that their performance is affected not only by the properties of these materials but often to an equal degree by the properties of the brick and reinforcement with which they are combined. For this reason, in selecting materials for use in a particular structure, the effect of the combined properties of all of the materials should be considered when the specifications are written for each material.

As an example, mortars of highest compressive strength do not develop highest tensile bond strength with all brick, also smooth textured brick develop slightly lower tensile bond strengths than rough textured brick. If bond between mortar and brick is a critical factor in the design, it may be desirable to specify a mortar of lower compressive strength than if compressive strength of the masonry is the critical factor.

Research which has been conducted on reinforced brick masonry during the past three decades has been directed toward establishing rational bases for design, toward the determination and evaluation of factors affecting the strength of reinforced brick masonry structures, and to determine ultimate strengths of reinforced brick masonry members which may be used as bases for working stresses.

In the sections which follow, data obtained from such tests will be summarized and conclusions will be stated. Limitations of space prevent a complete review of all tests discussed for which the reader is referred to the original reports.

The physical properties of brick, including size, color and texture, are reviewed in considerable detail in Chapters 3 and 4 of "Brick and Tile Engineering", published by the Structural Clay Products Institute in 1950. This discussion will not be duplicated here, but only those properties of brick which appear to have the most important effect upon the performance and durability of reinforced brick masonry structures will be considered. Similarly, the discussion of mortar and grout, covered more completely in Chapter 5 of Brick and Tile Engineering, will be limited to those mortars recommended for use in reinforced brick masonry.

202. BRICK

Of the materials—brick, mortar, grout and reinforcement—which compose reinforced brick masonry, the physical properties of brick produced in different localities probably vary more than the properties of either the mortar

ingredients or reinforcement. This is due to the fact that clays and shales are used for the manufacture of brick as they occur in nature and, for economic reasons, benefication of the clay or other treatments to produce a uniform raw material are impractical. The resultant product, therefore, will reflect the variations of the natural clays and, to a much greater extent than most other manufactured articles, may be said to be a natural product.

The properties of brick which have been found to have the most important effect upon the design, performance and durability of reinforced brick masonry are size, compressive strength, water absorption including saturation coefficient, and those properties affecting the bond between mortar and brick, such as the rate of absorption when laid and surface texture.

(a) Sizes of Brick. The American Standard Sizes of Clay and Concrete Modular Masonry Units, A62.3, approved in August 1946, lists modular sizes of clay and concrete masonry units. Since the adoption of this standard, additional modular sizes have been produced by the industry which can be laid economically in modular construction, but which would require excessive cutting if used in a non-modular design.

Table 2-1 lists the sizes of brick and Fig. 2-1 shows typical shapes currently available; however, few manufacturers produce all of these sizes and shapes and it is suggested that the purchaser ascertain the units available in any locality before proceeding with a design.

TABLE 2-1

NOMINAL MODULAR SIZES OF BRICK ①

Thickness, in.	Face Dimension in Wall	
	Height, in.	Length, in.
4	2	12
4	2⅔	8
4	2⅔	12
4	4	8
4	4	12
4	5⅓	8
4	5⅓	12

① Nominal sizes include the thickness of the standard mortar joint for all dimensions.

Nominal masonry unit dimensions are joint center line dimensions and differ from the specified unit dimension by the thickness of one mortar joint. Actual unit dimensions may vary from the specified dimensions by the permissible tolerances, plus or minus, included in the specifications.

Standard joint thicknesses for facing brick are ⅜ in. or ½ in., not both; that is, brick produced to lay with a ½-in. joint are not also produced to lay with a ⅜-in. joint. Standard joint dimensions for building brick are ½ in. and for glazed brick ¼ in.

For a more complete discussion of modular masonry, the reader is referred to Chapter 2 of Brick and Tile Engineering, SCPI, 1950.

(b) Compressive Strength. The compressive strength of brick has an important effect upon the compressive strength of reinforced brick masonry walls, columns and flexual members and, in combination with water absorption and saturation coefficient, is a measure of the durability of brick. Compressive strength is defined as the maximum resistance of the unit to a

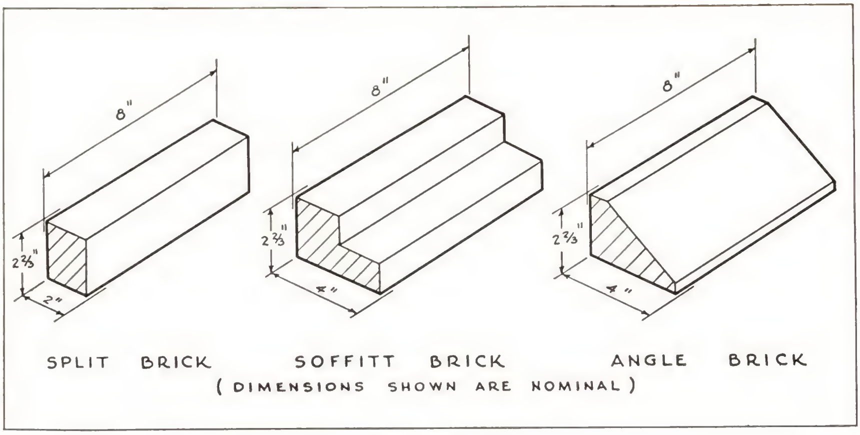


Fig. 2-1
Typical Shapes Used in Reinforced Brick Masonry

gradually increasing load applied at right angles to the plane of the bearing surface of the unit.

Test procedures are prescribed in ASTM Standard Methods of Sampling and Testing Brick, C67-, which provide that the test shall be made on a half brick tested flatwise; that is, the load is applied in the direction of the thickness of the unit.

Compressive strengths of clay and shale brick are reported in the paper, "Strength, Water Absorption and Weather Resistance of Building Bricks Produced in the United States", by J. W. McBurney and C. E. Lovewell, published in Vol. 33, Part II, of the Proceedings of the American Society for Testing Materials.

Table 2-2, taken from this paper, is based upon samples representing 37 per cent of the 1929 brick production (clay and shale) of the United States. As indicated, 92 per cent of the brick have flat compressive strengths of 3000 psi or better, excluding salmons. The weighted average of all brick, both hard and salmon, is 7246 psi; hard brick only, 7437 psi; salmon brick only, 4094 psi. From the distribution data given, approximately 6 per cent of the production is 1250 to 2500 psi, 20 per cent is 2500 to 4500 psi, and 74 per cent is 4500 psi or over. Approximately 40 per cent of the production is 8000 psi or over in compressive strength.

The raw materials used for the production of brick in 1929 are representative of the clays and shales used in current production; however, subsequent to that time, many manufacturers have improved their methods of manufacture, particularly through the adoption of deairing and modern methods of firing. The effect of these improvements is, in general, to increase the density of the unit, thus reducing absorption and increasing compressive strength. The effect of compressive strength of brick upon masonry strength will be discussed in Section 205(a).

(c) Water Absorption. The water absorption of a brick is defined as the weight of water, expressed as a percentage of the dry weight of the unit, which is taken up by the unit under a given method of treatment. Water absorption is determined by a number of methods, such as partially or totally

TABLE 2-2

DISTRIBUTION OF STRENGTH PROPERTIES OF BRICK FROM ALL PARTS OF THE UNITED STATES

Compressive Strength, Flatwise		Modulus of Rupture	
Range, psi	Percentage of production within range	Range, psi	Percentage of production within range
21001 to 22500	0.46	2101 to 3450	6.95
19501 to 21000	0.69	1951 to 2100	3.00
18001 to 19501	0.46	1801 to 1950	2.74
16501 to 18000	2.04	1651 to 1800	7.57
15001 to 16500	1.49	1501 to 1650	8.34
13501 to 15000	3.71	1351 to 1500	5.34
12001 to 13500	4.76	1201 to 1350	7.12
10501 to 12000	7.78	1051 to 1200	10.55
9001 to 10500	8.61	901 to 1050	10.44
7501 to 9000	11.92	751 to 900	13.60
6001 to 7500	15.47	601 to 750	11.74
4501 to 6000	16.81	451 to 600	7.52
3001 to 4500	17.97	301 to 450	4.35
1501 to 3000	7.46	151 to 300	0.37
0 to 1500	0.36	0 to 150	0.37
Total per cent....	99.99		100.00

immersing the unit in cold distilled water for various periods of time ranging from a few minutes up to several days; by boiling from 1 to 5 hr., or treating the unit, while immersed, to alternate applications of vacuum and pressure. All these methods fill the pores more or less completely with water. The boiling and vacuum treatments obviously give a more complete pore filling and, hence, a higher water absorption than the other methods.

Procedures for determining water absorption are included in ASTM Standard Methods of Sampling and Testing Brick C67-.

Absorption is obtained from the formula:

$$\text{Per cent absorption} = \frac{W - D}{D} \times 100$$

Where W = weight of specimen after immersion in water
D = dry weight of specimen.

Table 2-3, taken from the paper "Strength, Water Absorption and Weather Resistance of Building Brick Produced in the United States", by J. W. McBurney and C. E. Lovewell, previously referred to, indicates that 3.87 per cent of all brick tested had water absorptions less than 2.0 per cent when subjected to 5-hr. cold total immersion. By the same test, only 0.11 per cent exceeded 26.0 per cent absorption, and none in excess of 28 per cent absorption. When water absorption was determined by the 5-hr. boil method, 0.53 per cent of all specimens was under 2.0 per cent absorption and only 0.75 per cent fell within the limits 28.0 to 34.0 per cent. The individual minimum was 0.3 per cent.

TABLE 2-3

DISTRIBUTION OF WATER ABSORPTION AND C/B RATIO FOR BRICK FROM ALL PARTS OF THE UNITED STATES

Water Absorption				Ratio, 48-hr. Cold to 5-hr. Boiling Water absorption	
Range, per cent	Percentage of Production Within Range			Range	Percentage of Production Within Range
	5-hr. Cold	48-hr. Cold	5-hr. Boiling		
0 to 2.00	3.87	2.65	0.53	0.16 to 0.30	0.37
2.01 to 4.00	8.05	4.96	2.77	0.31 to 0.35	0.45
4.01 to 6.00	14.99	13.00	3.27	0.36 to 0.40	0.21
6.01 to 8.00	13.73	14.22	9.94	0.41 to 0.45	0.64
8.01 to 10.00	16.52	15.82	13.77	0.46 to 0.50	1.28
10.01 to 12.00	11.80	11.77	11.19	0.51 to 0.55	1.17
12.01 to 14.00	11.14	15.29	13.88	0.56 to 0.60	3.30
14.01 to 16.00	8.84	8.24	13.09	0.61 to 0.65	8.64
16.01 to 18.00	5.01	5.52	11.02	0.66 to 0.70	12.37
18.01 to 20.00	2.42	4.77	9.14	0.71 to 0.75	16.80
20.01 to 22.00	2.22	2.31	5.72	0.76 to 0.80	19.41
22.01 to 24.00	1.31	0.64	2.14	0.81 to 0.85	15.65
24.01 to 26.00	0.00	0.68	1.95	0.86 to 0.90	13.89
26.01 to 28.00	0.11	0.00	0.75	0.91 to 0.95	5.32
28.01 to 34.00	0.00	0.11	0.75	0.96 to 1.00	0.51
Total per cent . . .	100.01	99.98	99.91		100.01

As previously indicated, improvements in manufacturing methods during the past 20 years have tended to reduce the absorption of brick produced by many manufacturers.

(d) Capillarity and Suction. The pores or small openings in burned clay products function as capillaries which tend to draw water into the unit. This action in a brick is referred to as its rate of absorption or suction.

Suction of brick is determined by partial immersion of the unit to a depth of ⅛ in. in water for a period of 1 min. The method of conducting this test is included in ASTM Sandard Methods of Sampling and Testing Brick, C67-.

Suction may be calculated by the following formula:

$$S = \frac{W' - W}{A} \times 30$$

where S = suction, in grams per minute per 30 sq. in.

W = weight of unit prior to partial immersion, in grams

W′ = weight of unit after partial immersion for 1 min., in grams

A = net area of surface of unit immersed, in square inches.

Note: 1 gram = 0.035 oz. approximately.

Numerous tests of the tensile strength of bond between mortars and brick indicate that suction has an important effect upon tensile bond strength upon which the transverse strength and watertightness of unreinforced brick walls largely depend.

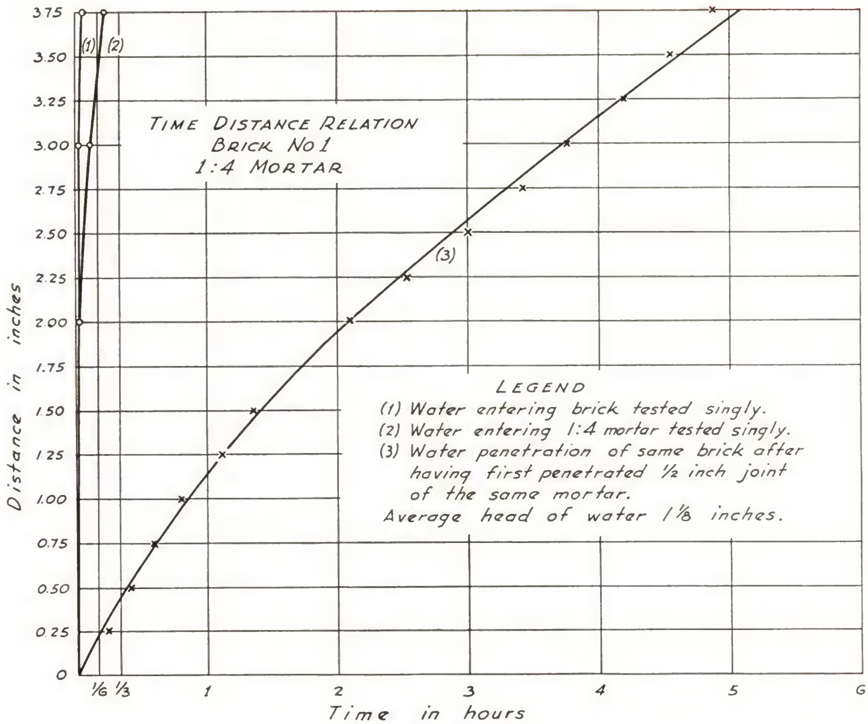


Fig. 2-2

Rates of Water Penetration Through the Separate Materials, Brick and Mortar, and Through a Brick in an Assemblage of Brick Bonded Together with Mortar

Suction of brick produced commercially vary from 1 to 2 grams to 60 or more grams. However, suction of units may be reduced to any desired value by wetting prior to laying or before the application of plaster or stucco.

There is no consistent relationship between total absorption and suction. Some brick having relatively low absorptions have high suctions and the reverse is also true.

Many brick of high suction will wet through from end to end within a few minutes when the brick is placed on end in about 1 in. of water. Moisture transferred in this manner by capillarity may be transmitted to plaster applied directly to the masonry and cause staining or softening. However, capillary action is not a source of wall leakage since free water is not transmitted through the brick, but, in practically every case, through openings between mortar and brick.

Tests by L. A. Palmer, reported in his paper, "Water Penetration Through Brick-Mortar Assemblages", published in the September 1931 Journal of the Clay Products Institute, indicate that the time required for the transmission of moisture through assemblages of high suction brick, in which the brick are separated by a mortar joint, is increased many times that required for moisture to travel an equal distance by capillarity through either the brick or the mortar.

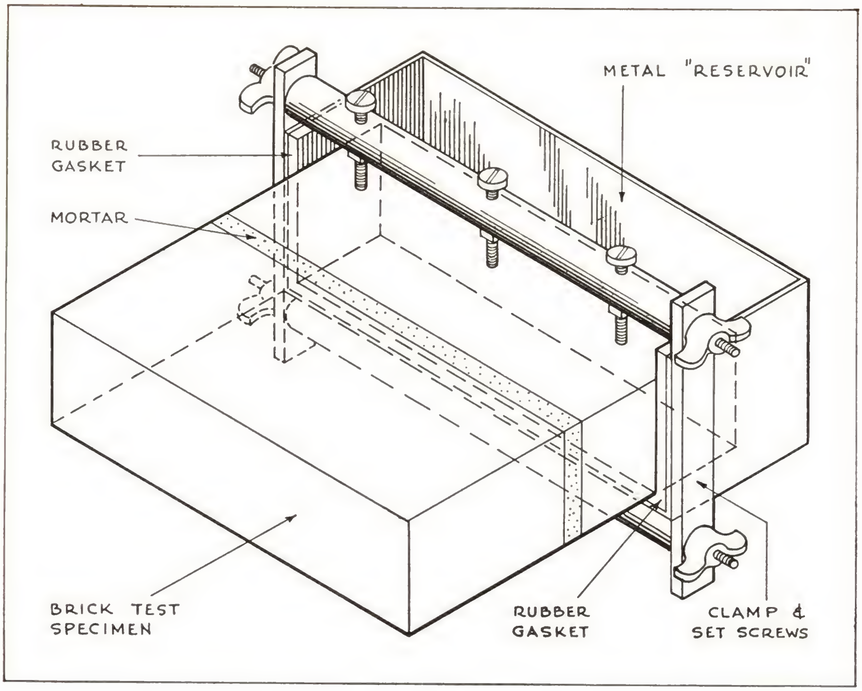


Fig. 2-3
Apparatus for Maintaining a Definite Exposure on a Definite Surface Area of Brickwork to Water at Constant Level

Fig. 2-2, reproduced from Palmer's paper, shows this relationship for a brick of high suction.

The test assembly used by Palmer to determine the rate of water penetration by capillarity consisted of 2 brick bonded together, edge to edge, with a ½-in. mortar joint as illustrated in Fig. 2-3. A metal reservoir was attached to the edge of one brick and a constant head of 1⅛ in. of water was maintained in this chamber. The assemblage is representative of a section of an 8-in. wall with the mortar joint representing the collar joint.

The brick from which the data in Fig. 2-2 were obtained were dry press having total absorptions ranging from 17 to 24.5 per cent.

Regarding the test, the author states: "Water traveled through the first brick of the assemblage (data not plotted) whose edge formed a side of the rectangular reservoir in about 5 min. There was a delay then of about 20 min. before the joint became noticeably wet in any part. At the end of a half hour the joint was wetted through. There was then a delay of about 15 min. before there was any visible sign of water entering the second (outer) brick. As it traveled through this brick, the times for wetting through each quarter of an inch were recorded. Water was 'fed' to this brick slowly and the rate of flow through it in the assemblage is represented by curve 3."

As a result of the tests reported in this paper, the author concludes: "It is improbable that water will be transmitted through solid 8-in. walls of brick if there are no open spaces between mortar and brick and if some

provision is made in so designing the wall that header brick do not connect directly with the interior. This may be had by furring the inner wall or by the use of metal ties in place of headers."

While capillary action does not contribute to wall leakage, it is recommended that, where plaster is desired for the interior finish of solid masonry walls containing through headers, it be applied over lath and furring rather than directly to the masonry.

(e) Texture. Texture is defined as the surface effect or appearance of the unit apart from its color. It may result from the normal process of manufacture, such as the smooth surface (die skin) which is produced by extruding the clay column through a steel die or the sand finish texture obtained in the soft mud process when the molds are lubricated with sand. Other textures are produced by mechanical means to obtain desired effects and range from very coarse, produced by scratching and rolling the clay column, to very fine where the die skin is removed by wire cutting.

Since it is practically impossible to describe characteristic textures in recognizable terms, specifications usually provide illustrations or samples (panels) of the texture desired and require that the product submitted shall meet the approval of the purchaser.

The more common textures available throughout the United States are illustrated in Chapter 3 of Brick and Tile Engineering, SCPI, 1950, and, as previously stated, have been developed to create a desired appearance.

Texture which affects the performance of reinforced brick masonry is the surface in contact with mortar and grout. These surfaces will usually be smooth, scored, combed or roughened, and are defined as follows:

Smooth Finish. Surfaces not altered or marked in manufacture but left a plane surface as formed by the die or mold.

Scored Finish. Face surfaces grooved as they come from the die to give increased bond for mortar, plaster or stucco.

Combed Finish. Face surfaces altered by more or less parallel scratches or scarfs in manufacture to give increased bond for mortar, plaster or stucco.

Roughened Finish. Face surfaces entirely broken by mechanical means, such as wire cutting or wire brushing, to give increased bond for mortar, plaster or stucco.

The effect of surface texture on bond will be discussed in Section 203(c).

(f) Durability. The weathering action that has the greatest effect on most building materials is alternate freezing and thawing in the presence of moisture. This is practically the only action of weathering that has any significant effect upon burned clay products.

Experience has indicated that any well burned brick will resist the action of freezing and thawing over a long period of time and, from a structural standpoint, may be considered durable.

There is a reasonably close correlation between the performance of structural clay products in the freezing and thawing test and under the agents of weathering in masonry structures and, at the present time (1953), this test appears to be the best measure of the durability of brick.

Freezing and thawing tests consist of subjecting the unit (or portions of it) to from 50 to 100 alternate cycles of freezing and thawing in the presence of moisture. This requires a period of over 10 weeks and makes the freezing and thawing test impractical as an acceptance test. For this reason, extensive

research has been carried on to correlate other physical properties of brick with their resistance to the freezing and thawing test.

For brick produced of the same raw material and by the same method of manufacture, either compressive strength or total absorption may be taken as a fairly accurate measure of the resistance of such brick to the freezing and thawing test; however, limits on these properties that apply to one product do not apply to products produced from different raw materials or by different manufacturing processes, and consequently they alone cannot be used as measures of durability in general specifications. A third property, known as "saturation coefficient", when used in conjunction with compressive strength and total absorption by 5-hr. boiling has been found to provide a means of predicting the resistance of most types of brick to freezing and thawing tests with greater accuracy than any other method developed to date (1953).

ASTM Specifications C62- and C216- for Building Brick and Facing Brick, respectively, prescribe limits on compressive strength, absorption and saturation coefficient as a measure of durability and the explanatory note appended to Specifications C62- includes a discussion of the grading of brick based on durability.

203. MORTAR AND GROUT

Perhaps the most important property of mortar affecting the performance of reinforced brick masonry is its ability to form a strong and durable bond with the brick and reinforcement. Other important properties are workability, compressive and tensile strength, durability and volume change under varying conditions to which it may be subjected. Of the cement-lime mortars in common use, none will rate highest in all of these properties. In selecting a mortar, therefore, for any particular use, it is necessary to evaluate the properties of various mortars with a view to use requirements.

ASTM Tentative Specifications for Mortar for Unit Masonry, C270-, include 5 types of mortar. Two alternate methods of specifying these mortars are provided: A Property Specification in which, quoting from C270-, "the acceptability of the mortar is based on the properties of the ingredients (materials) and the properties (water retention and compressive strength) of samples of mortar mixed and tested in the laboratory"; and a Proportion Specification in which "the acceptability of the mortar is based on the properties of the ingredients (materials) and the proportions of these ingredients."

Mortars recommended in Chapter 3 for use in reinforced brick masonry are portland cement lime mortars meeting the requirements of ASTM Tentative Specifications C270- for types A-1 and A-2. The property specification for these mortars provides that the volume of aggregate shall be not less than 2¼ nor more than 3 times the total separate volumes of cementitious materials, that the flow after suction shall be not less than 70 per cent, and that the compressive strength of 2-in. mortar cubes cured in water and tested at 28 days shall be not less than 2500 psi and 1800 psi for types A-1 and A-2, respectively.

The proportion specification fixes the proportions of cement, lime and aggregate within limits as indicated in Table 2-4.

TABLE 2-4

MORTAR PROPORTIONS BY VOLUME

Mortar Type	Parts by Volume of Cement	Parts by Volume of Hydrated Lime or Lime Putty	Aggregate, measured in a damp loose condition
A–1	1	¼	Not less than 2¼ and not more than 3 times the sum of the volumes of the cement and lime used.
A–2	1	over ¼ to ½	

The property specification of ASTM Specifications C270— covers mortars in which the cementitious material is masonry cement, as well as cement and lime, and a recent amendment to the proportion specification fixes proportions of portland cement and masonry cement for A-1 and A-2 mortars. However, due to the uncertainty as to the admixtures contained in masonry cements and because of the effect of certain types of admixtures on bond between mortar and brick and between mortar and reinforcement as discussed in Section 203(h), the use in reinforced brick masonry of mortar containing masonry cements is not recommended unless laboratory data or performance records are available which indicate that such mortar produces satisfactory bond with both brick and reinforcement.

(a) Workability. A workable mortar is obviously essential to good workmanship. While there is no single satisfactory measure of workability, it is related to both mortar flow and water retentivity. Both of these are measured on the flow table which is a plane table so constructed that it can be dropped through a height of .5 in. by means of a rotating cam. This test has been subjected to criticism by many investigators, since results are affected both by the condition of the apparatus (mounting and wear of movable parts) and by the technique of the operator so that frequently results are not reproducible on different flow tables by different operators.

The variability of flow of mortars, as determined by different laboratories, is discussed at considerable length in the appendix to the 1942 Report of ASTM Committee C-12. This appendix is a report by J. W. McBurney, then chairman of Committee C-12, of the results of cooperative tests on masonry mortars by 14 laboratories.

After reviewing the data presented by these laboratories, the report states: "There is evidence that one laboratory's flow of 105 per cent is another laboratory's 135 per cent, and yet another's 70 per cent."

Subsequent to the publication of this report, the Bureau of Standards has developed a standard mixture for use in calibrating flow tables. Through the use of this mixture, correction factors or curves can be established for individual flow tables by means of which flows obtained on these tables can be converted to values comparable to those obtained on the standard table at the Bureau of Standards.

While absolute flows obtained in different laboratories are not always comparable, flow after suction, which is a ratio of two flows, is much less subject to variation.

Mortar flow is defined as the per cent increase in the diameter of a conical frustum of mortar, 4 in. in diameter at its base, after the flow table has

been dropped through a height .5 in. 25 times in 15 sec.; that is, if the diameter of the mortar mass is 8 in. after the test, the flow is 100 per cent.

Effect of Lime. Water retentivity is defined as the ratio of the flow of the mortar, after being subjected to a partial vacuum for 1 min., to its initial flow; that is, if the flow of the mortar after suction is 84 per cent and the initial flow was 120 per cent, the water retentivity is the ratio or 70 per cent.

Methods of determining flow and performing the flow after suction test are described in ASTM Specifications for Masonry Cement, C91-.

Water retentivity of mortars is affected by the ingredients, both cementitious and aggregate, and for a given proportion may be increased through the use of highly plastic lime putties.

Research conducted at the National Bureau of Standards and reported in Research Paper No. 952, "Differences in Limes as Reflected in Certain Properties of Masonry Mortars", by Lansing S. Wells, David L. Bishop and David Watstein, indicates that there is a wide variation in the plasticity and water retaining power of commercial hydrated limes; such a variation, in fact, that the investigators state: "The flow after suction of cement-lime mortars depends far more on the properties of the lime than on the cement-lime ratio."

Fig. 2-4, reproduced from Bureau of Standards Research Paper RP952, shows the effect on the flow after suction of a cement mortar (1 part cement,

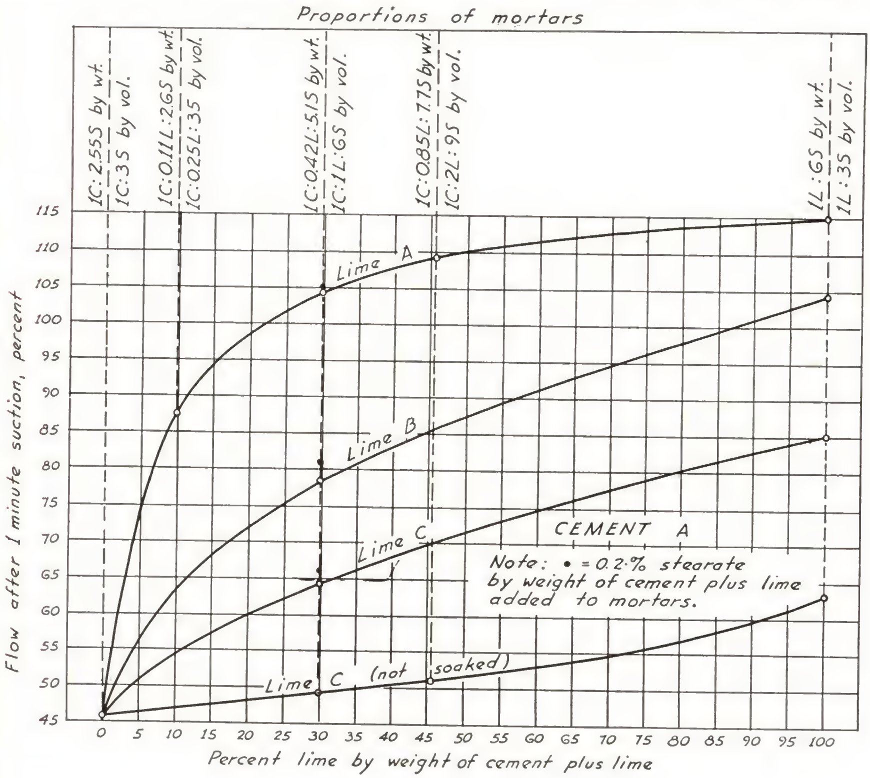


Fig. 2-4

The Effect of Additions of Lime on the Flow After Suction of a Cement Mortar

3 parts sand by volume) of the additions of varying amounts of lime putties prepared from three different limes, and similar additions of one hydrated lime not soaked.

The flows after suction, shown in Fig. 2-4, are absolute flows; that is, the increase in diameter of the mortar mass, expressed as a percentage of the original diameter of 4 in.; and should not be confused with water retentivity which is the ratio of the flow after suction to the initial flow. The initial flow of the mortars, included in Fig. 2-4, was 130 per cent; consequently, to obtain water retentivity, the flows after suction listed should be divided by 1.3 to obtain per cent of initial flow.

Regarding the data plotted (Fig. 2-4), the authors of Research Paper RP952 conclude: "The flow after suction of lime mortars is dependent on the plasticity of the putty used in preparing mortar." It will be noted also from Fig. 2-4 that the addition of 0.2 per cent stearate increased the flow after suction but slightly, even though such additions appear to increase the workability of the mortar.

Workable mortars, as judged by a skilled mason, will usually have flows from 110 to 140 per cent and, in general, mortars of best workability have high water retentivity.

Mortar flow has an important effect on both mortar strength and bond between mortar and brick. These effects are discussed in Sections 203(b) and 203(c), respectively.

(b) Strength. The tensile strength of standard mortar briquets and the compressive strength of 2-in. cubes or 2-in. by 4-in. cylinders are usually taken as measures of mortar strength. Test specimens are cast in non-absorbent molds and may be cured in air, in a moist closet, in water or by some combination of these conditions. For hydraulic mortars, curing in water is usually specified, due to the ease of maintaining uniform curing conditions for different determinations.

Standard methods of forming briquets and cubes are described in ASTM Specifications C150- and C91- for portland and masonry cements, respectively.

As in the case of concrete, the strength of masonry mortars depends to a large degree on the amount of portland or hydraulic cement which they contain and upon the water-cement ratio. However, since in most masonry structures strength of mortar is of secondary importance to bond strength, workability, water retentivity and low volume change, these properties are usually given primary consideration in specifying mortars.

While strength of mortar, per se, is not of primary importance in most masonry structures, it has a relation to other properties of mortar and, for similar ingredients, is a reasonably accurate check on the proportioning and mixing. The determination of strength can be made relatively simply and the standard test methods give consistent and reproducible results. For these reasons, strength is usually considered an important basis for comparing mortars.

Effect of Proportion and Flow. Tables 2-5 and 2-6 are compiled from data obtained from a series of tests of "Tension Bond Between Mortar and Brick", performed in 1942 by Robert S. McBurney, Research Associate in Mechanics of the University of Wisconsin. This program was sponsored by the Structural Clay Products Institute and tests were made under the supervision of Professor M. O. Withey, now (1953) Dean of Engineering, University of Wisconsin.

The program included tension bond tests of 467 specimens, and briquet and cube tests of 279 specimens each. Mortars 1, 2 and 3 in Table 2-5 are composed of similar ingredients but of different proportions. The lime was a lime putty weighing 77.95 lb. per cu. ft. and contained 37.5 per cent by weight of lime solids.

TABLE 2-5

COMPRESSIVE AND TENSILE STRENGTHS OF MORTARS

Mortar No. and Mix ①	Initial Flow %	Flow after Suction, % of Initial Flow	Strength, psi, ②	
			Tension	Compression
1 1:¼:3	100 120 133	87 87 87	457 425 420	5492 5153 4830
2 1:½:4½	100 120 133	89 88 88	300 277 268	2758 2408 2175
3 1:1:6	100 120 133	92 93 91	180 165 145	1173 905 793

① Proportions: Cement, lime, sand by volume.
② Tension specimens, briquets; compression, 2-in. cubes; both cured in water, tested at 28 days.

Mortar 2 of Table 2-6 is the same as mortar 2 of Table 2-5, and mortars 4 and 5 are of the same proportion: 1 part cement, ½ part lime and 4½ parts sand by volume. However, the lime used in mortar 4 was a pressure hydrated dolomitic hydrate and the lime used in mortar 5 was a commercial mason's hydrate.

TABLE 2-6

COMPRESSIVE AND TENSILE STRENGTHS OF MORTARS

Mortar No. and Mix ①	Initial Flow %	Flow after Suction, % of Initial Flow	Strength, psi, ②	
			Tension	Compression
2 1:½:4½	100 120 133	89 88 88	300 277 268	2758 2408 2175
4 1:½:4½	100 120 133	80 80 80	350 343 347	3488 3215 2938
5 1:½:4½	100 120 133	63 64 66	348 358 348	3642 3503 3290

① Proportions: Cement, lime, sand by volume.
② Tension specimens, briquets; compression, 2-in. cubes; both cured in water, tested at 28 days.

Values listed in Tables 2-5 and 2-6 are averages of from 17 to 21 specimens.

Table 2-5 shows the relation of strength of mortar to the relative proportions of cement and lime. While absolute strengths of mortars of similar proportions will vary, depending upon the properties of the ingredients, particularly the lime, the trend shown in Table 2-5 is typical. Highest strength is associated with high proportion of cement.

The relation of water retentivity to proportion, shown in Table 2-5, is also typical. For the same ingredients, higher proportions of lime are associated with increased water retentivity; however, as indicated in Section 203(a), this relationship may not hold for different materials, particularly different limes.

As would be expected, increased flows which produce increased water-cement ratios are associated with decrease in strength.

The data in Table 2-6 illustrate the effect of different limes on water retentivity and strength. In general, mortars containing lime putty have lower strengths than those containing hydrates. Mortars of high water retentivity have lower strengths than mortars of the same proportion with lower water retentivity.

Effect of Age and Shape. Table 2-7 is compiled from data included in a report to the Clay Products Institute of California by Raymond E. Davis, giving the results of tests "to determine the flexual, adhesive, shearing and bond strengths of brick and hollow tile masonry." These tests were made at the University of California under Professor Davis's direction during the period October 1929 to December 1930. The entire program included approximately 1900 individual tests and is one of the most extensive investigations of the factors affecting the flexural strength of unreinforced masonry that has been reported.

TABLE 2-7
STRENGTHS OF MORTARS

Mortar No. and Mix ①	Initial Flow %	Tensile Strength, psi					Compressive Strength, psi				
		7 days	28 days	2 mos.	6 mos.	12 mos.	7 days	28 days	2 mos.	6 mos.	12 mos.
Series 1											
No. 1 1:⅓:4	196	231	267	420	416	516	1530	2660	3100	2930	3570
No. 2 1:½:4½	179	237	310	362	325	496	1436	2340	2480	2470	3180
No. 3 1:1:6	174	118	200	294	244	304	674	1335	1790	1700	1925
Series 2											
No. 3 1:1:6	.....	.....	180	207	.....	.....	......	900	1099	....	....
No. 3 1:1:6 Dry cure	.....	.....	71	110	.....	.....	......	485	706	....	....

① **Proportions:** Cement, lime, sand by volume.

Table 2-7 shows the tensile and compressive strengths of mortars of 3 different proportions when tested at 7 days, 28 days, 2 months, 6 months and 12 months. Similar ingredients were used in all mortars, except that a different lime was used in the mortar of series 2. In all mortars the lime consisted of lime putty prepared from a high calcium quicklime. Tension tests were made on standard mortar briquets and compression tests on 2-in. by 4-in. cylinders. The values listed are averages of 6 specimens.

As indicated in Section 203(a), flows of mortars obtained on different flow tables by different operators are frequently not comparable, and it is quite probable that the flows reported in Table 2-7 correspond to flows of 130 to 140 as reported in the University of Wisconsin data and tabulated in Table 2-5.

Regarding the curing of the specimens, the report states: "Except as otherwise noted, all mortar and masonry specimens were wet twice daily with a hose for the first 14 days; specimens tested at 6 months or earlier were then left in air until tested. Those tested at 12 months were left in air following the 14-day wetting period, until 8 months of age, after which they were wet with a hose once daily for 7 consecutive days at the beginning of each month, the last wetting being 3 weeks before the time of testing. All were stored in air at about 60° F and 65 per cent relative humidity."

The differences between the strengths reported in Table 2-5 and Table 2-7 of mortars of approximately the same proportions are due not only to the different ingredients of the mortars but also to different methods of curing and the shape factor of the compressive test specimens.

ASTM Specifications C42-, Standard Methods of Securing, Preparing and Testing Specimens from Hardened Concrete for Compressive and Flexural Strengths, provide that, when the ratio of height to thickness (h/d) of a compressive test specimen is less than 2, the strength shall be multiplied by a correction factor to give a value comparable to the strength obtained from a specimen having an h/d ratio of 2. This correction factor for an h/d ratio of 1 (cube) is .85. The relation of strength of mortar to relative proportions of cement and lime, shown in Table 2-7, is similar to that indicated in Table 2-5; however, the former also shows the effect of age on strength and, to a degree (series 2) the effect of curing conditions.

In general, for the mortars tested, strength increases with age up to 12 months and mortar specimens cured in air have lower strengths than similar specimens wet periodically during the curing period. Specimens of hydraulic mortars cured in water, as a rule, give highest strengths.

Effect of Retempering. Table 2-8 is compiled from data included in "Report of Tests", by Robert R. Schneider, Department of General Engineering, University of Southern California, dated November 1951. This work was sponsored by the Associated Brick Manufacturers of Southern California and included tests on reinforced brick beams and grouted wall panels, as well as supplementary tests.

The mortar, from which the data reported in Table 2-8 were obtained, consisted of 1 part cement, 3/10 parts hydrated lime and 4½ parts sand by volume. The lime was a pressure hydrated dolomitic hydrate. Tension test specimens were standard briquets, and compressive test specimens were 2-in. by 4-in. cylinders. All specimens were cured in a moist room and tested at the age of 28 days. The mortar consistency was that judged by the mason to be satisfactory.

In molding the specimens, the briquet molds and cylinder molds with bottoms removed were placed on blotting paper laid on top of a brick. The suction of the brick removed some of the water from the mortar, thus reducing the water-cement ratio. It was believed that this procedure "approximates job conditions in the mortar specimens."

While it is probably true that the mortar strengths obtained by this method of forming more nearly approximate the strengths developed in masonry construction than the strengths obtained from specimens molded in non-absorbent molds, the above procedure introduces an uncontrolled variable into the test which makes comparison with other data difficult.

Table 2-8 shows the effect on mortar strength of remixing and retempering at various intervals after first mixing. The treatment given the mortar is described in the report as follows:

"Tests were made at the following times after mixing: 15 min., 1 hr. (mortar remixed with a trowel, no water added), 2 hr. (mortar remixed with a trowel, no water added), 3 hr. (water added 10 min. previous to test and remixed to job consistency), 4 hr. (water added 1 min. previous to test and remixed to job consistency)."

TABLE 2-8

EFFECT OF RETEMPERING ON MORTAR STRENGTH

Time after mixing	Initial Flow %	Flow after Suction, % Initial Flow	Strength, psi	
			Compression	Tension
15 min.	99	92.1	3473	433
1 hr.	86	82.3	3406	....
2 hr.	76	83.7	2988	....
3 hr.	88	94.0	2577	....
4 hr.	95	87.0	2006	268

As indicated in Table 2-8, strength of mortar decreases due to retempering as the time after original mixing increases. It will be noted that decrease in compressive strength after retempering at the end of 3 hr. is approximately 26 per cent of the 15-min. strength, and it is probable that this reduction would be somewhat less at the end of 1 hr. and 2 hr., even if the mortar had been retempered through the addition of water. Since, as will be discussed in Section 203(c), increased flow of mortar increases bond strength between mortar and brick, it is frequently desirable to sacrifice some mortar strength in the interest of improved bond.

Effect on Masonry Strength. Compressive strength of mortar is related to compressive strength of masonry; however, the effect of mortar is more nearly in proportion to the cube root of its strength than directly proportional.

Tensile strength of mortar may affect the lateral strength of unreinforced masonry walls, provided bond strength between mortar and brick exceeds the tensile strength of the mortar. However, for mortars specified for reinforced brick masonry, this condition will rarely prevail and the bond strength will usually be the critical factor.

It is recognized that mortar specimens formed in masonry molds develop substantially higher strengths than when formed in non-absorbent molds

and it is probable that the former more nearly approximate the strength developed in masonry. However, since, as previously stated, mortar strength is rarely a critical factor in design, the current standard methods of measuring strength are recommended, due to the simplicity and reproducibility of the test method.

(c) Bond–Mortar to Brick. In the synopsis of the paper, "Measurement of Bond Between Bricks and Mortar", by J. C. Pearson, published in the 1943 Proceedings of the American Society for Testing Materials, Vol. 43, page 857, the author states: "Experience with various masonry bond tests has shown that most of them give discouragingly wild results and that good reproducibility is obtained, if at all, by artificial procedures which neither imitate the bricklayer's manipulations nor produce results anything like those obtained in practical masonry construction."

Anyone who has studied the data obtained from bond tests reported by various investigators will confirm Mr. Pearson's statement. The wide variations between individual specimens of the same group frequently encountered in such data are due to the large number of variables, some of which are very difficult to control, to which all tests, which might be expected to produce results comparable to those obtained in masonry construction, are sensible.

These variables include flow of mortar, elapsed time between spreading mortar and placing brick in contact with it, suction of brick, pressure or tapping applied to joint during forming, texture of brick surface and probably, to a minor degree at least, other factors which have not been identified.

In the paper, "Measurement of Bond Between Bricks and Mortar", the author reports data obtained from a number of different methods of test to determine bond between mortar and brick which were made in an attempt to develop an improved test method "imitating job conditions and manipulations without involving too many uncontrolled variables". As a result of this study, a "mechanized cross-brick couplet assembly" is proposed as the most satisfactory test specimen for determining bond between mortar and brick.

While the test methods proposed by Pearson give more uniform results than most other methods that attempt to approximate field conditions, the number of test specimens required to obtain a reasonable average is large and many of the variables listed above are still difficult to control. For these reasons, bond tests, either tensile or modulus of rupture, are not considered satisfactory as acceptance tests for mortar. They are, however, of value in comparing the relative bonding properties of mortars and, within limits, in providing an indication of the bond developed in masonry construction between mortar and brick.

Effect of Mortar Flow. Table 2-9 is reproduced from data obtained in the University of Wisconsin tests described in Section 203(b). Mortars 1, 2 and 3 are of similar ingredients, and the lime was a lime putty weighing 77.9 lb. per cu. ft. and containing 37.5 per cent by weight of lime solids. Mortar 4 contained a pressure hydrated dolomitic lime and mortar 5 a commercial mason's hydrated lime.

Tensile bond test specimens were cross-brick couplets cured in air and tested at the age of 28 days. All values reported are the average of 5 to 15 specimens, in most cases 10.

TABLE 2-9

TENSILE BOND STRENGTH OF MORTARS

Mortar No. and Mix①	Initial Flow, %	Flow After Suction, % of Initial Flow	Compressive Strength, psi	Tensile Bond Strength, psi — Brick Suction, grams 5–10	10–20	20–40
1	125–135	87	4830	49	76	45
1:¼:3	110–125	87	5153	49	49	10
	95–110	87	5492	18	11	2
2	125–135	89	2175	54	77	67
1:½:4½	110–125	88	2408	43	58	44
	95–110	88	2758	22	23	14
3	125–135	92	793	47	72	59
1:1:6	110–125	93	905	49	56	53
	95–110	91	1173	14	36	23
4	125–135	80	2938	64	62	56
1:½:4½	110–125	80	3215	41	46	34
	95–110	80	3488	28	21	6
5	125–135	63	3290	66	70	49
1:½:4½	110–125	64	3503	70	57	32
	95–110	66	3642	50	44	8

① Proportion: Cement, lime, sand by volume.

Note: 1 gram equals 0.035 oz. approximately.

As may be noted from Table 2-9, for the mortars reported (all relatively high strength mortars), there is no consistent relationship between mortar strength and tensile bond strength. However, the relation between mortar flow and tensile bond strength is quite apparent; bond strength increasing as flow increases for all mortars and, with minor exceptions, for all brick suctions.

This effect of mortar flow on tensile bond strength has been observed by many investigators. In the paper, "Measurement of Bond Between Bricks and Mortar", Pearson states: "It is known, however, that the consistency of the mortar must be closely controlled. As evidence of this, a certain mortar was gaged to a 3¼-in. slump at the start of making a set of couplets and when the first 10 were completed the slump had fallen to 3 in. With the same mortar, another set of 10 couplets was made, starting with a slump of 2¾ in. and ending at 2⅝ in. The average tensile strength for the first ten was 68 psi, for the second ten 44 psi. This experience indicates that the slump should be maintained within ¼ in. of the desired value if it is not to have considerable effect on the bond strength."

The substantial increase in tensile bond strength with increased mortar flow, indicated in Table 2-9 and reported by Pearson and others, indicates the importance of controlling this variable in assembling tensile bond specimens, and also of maintaining a high flow in mortar used in masonry construction.

The time between spreading a mortar joint on one brick and placing a brick on this joint obviously will affect the flow of the mortar when it comes in contact with the top brick, particularly if the mortar is spread on a brick

of relatively high suction when placed in the wall. For highest bond strength, this time interval should be reduced to a minimum.

Effect of Retempering. Loss of water from the mortar, after mixing, by evaporation will decrease the flow and thus reduce tensile bond strength. The effect on mortar strength of remixing with the addition of water (retempering) at various periods after the mortar was first mixed is discussed in Section 203(b).

In the tests reported by Pearson ("Measurement of Bond Between Bricks and Mortar"), the practice was adopted of retempering the mortar after the first 10 of a group of 20 cross-brick couplets had been assembled. Regarding the effect of this retempering on tensile bond strength, the author states: "If retempering affects the bond strength materially, it should cause the average of the first 10 couplets of a set to be higher or lower than the average of the second 10 in the long run. The grand average difference from all the sets listed is less than 2 per cent, in favor of the first 10."

It appears, therefore, that the effect of retempering in reducing tensile bond strength is much less than the effect of decreased mortar flow.

Effect of Brick Suction. Table 2-9 also shows the relation of brick suction to tensile bond strength. For high flow mortars (125 to 135 per cent), maximum bond strength is obtained with the 10 to 20 gram suction (mortar 4 is an exception). The same is true for flows of 110 to 125 per cent, but the difference between the 10 to 20 gram range and the 5 to 10 gram range is not so great as for the higher flow mortars. For low flow mortars, the highest tensile bond strength is with brick of 5 to 10 gram suction, with the exception of mortar 3 which has an unusually high water retention; flow after suction 92 per cent.

These data emphasize the importance of wetting brick of high suction prior to laying. It will be noted that in all cases (all mortars and all flows) tensile bond strength is greater with brick suction of 10 to 20 grams (.35 to .70 oz.) than with brick suction of 20 to 40 grams (.70 to 1.4 oz.). This is true even with mortar 3 of extremely high water retentivity (flow after suction 92 per cent), and for mortars of lower water retentivity, such as mortar 5 (flow after suction 64 per cent), the decrease in bond strength for the higher suction brick is much greater. For brick suctions exceeding 60 grams (2.1 oz.), tensile bond strength may be reduced to a very low value.

Effect of Grout. In the investigation at the University of Wisconsin to determine plastic flow of reinforced brick masonary, discussed in Section 206(d), the program included tensile bond tests of brick couplets constructed of the brick and both the mortar and grout used in building the plastic flow beams.

Two types of brick were used, designated as "Streator" and "Chicago". The suction of the Streator brick ranged from 1.4 grams to 15.9 grams, and the average suction of the Chicago brick was 49.1 grams. The Chicago brick were wet prior to assembling the tensile bond specimens.

Two types of mortar and grout were used. Type A mortar consisted of 1 part cement: ¼ part hydrated lime; 3 parts sand by volume; and Type C mortar consisted of 1 part cement: 1 part hydrated lime: 6 parts sand by volume. Grouts were of the same proportion as the corresponding mortar with sufficient water added to produce pouring consistency.

The grouted test specimens were assembled in a form and the grout joint was in a vertical position until the grout stiffened.

TABLE 2-10

BOND TESTS ON GROUTS

Brick Type	Position	Grout from Mortar	Area, sq. in.	Load, lb.	Stress, psi
Streator	Bed	A	15.15	1450	96
			15.72	1625	103
			13.32	2011	151
					Av. 117
Streator	Bed	C	13.14	812	62
			12.78	931	72
			13.32	1228	92
					Av. 75
Chicago C	Bed	A	13.68	690	41
			13.68	932	56
			14.25	660	46
					Av. 48
Chicago C	Bed	C	13.32	1150	84
			13.68	440	32
					Av. 58
Streator	Face	A	6.44	630	98
			5.50	195	36
			6.00	360	60
					Av. 65
Streator	Face	C	5.28	164	31
			5.63	322	57
			5.40	324	64
					Av. 51
Chicago C	Face	A	6.00	442	74
			5.75	80	134
			5.63	928	165
					Av. 124
Chicago C	Face	C	5.52	315	57
			5.28	974	184
			5.06	525	104
					Av. 115

Twenty-three tensile bond test specimens formed with grout and 24 specimens formed with mortar were tested. Results of the mortar specimen tests were very erratic and inconclusive; however, the results obtained from the grout specimens were quite consistent and indicate higher tensile bond strengths than are generally reported for mortared specimens.

Table 2-10 gives the ultimate tensile strength of the grouted specimens.

These data are of particular interest since few tests to determine bond between brick and grout have been reported. It will be noted that tensile bond strength developed between the grout and the bed of the Streator brick averaged 117 psi for A grout in spite of the low suction, 1.4 to 15.9 grams of the Streator brick.

The decrease in tensile bond stress between the bed and the face of the Streator brick is explained in the report: "The bond was greater between the beds of the Streator brick in all grouted specimens than between the faces because the Streator brick are side-cut and have an almost glazed skin on the face."

Tensile Strength vs. Modulus of Rupture. Tables 2-11 and 2-12 are compiled from data included in the report to the Clay Products Institute of California by Raymond E. Davis, reviewed in Section 203(b). All mortars are of similar ingredients with the exception of mortar 3, series 2, in which a different lime was used. The lime for all mortars consisted of a lime putty prepared from high calcium lump lime.

TABLE 2-11

TENSILE BOND STRENGTH OF MORTARS

Mortar			Tensile Bond Strength③, psi Brick Designation and Absorption④, %		
No.	Mix①	Compressive Strength, psi②	A 12.0	B 13.6	C 8.6
1	1:⅓:4	2930	65	43	85
2	1:½:4½	2470	81	87	65
3	1:1:6	1700	33	28	49

① Cement, lime, sand by volume.
② Two-inch by 4-in. cylinders, age 6 months.
③ Cross-brick couplets, age 6 months.
④ Absorption by 48-hr. immersion cold water.

Mortars were mixed to a consistency satisfactory to the mason which gave average slumps, using the 6-in slump cone, of 3¾, 3¾ and 3¼ in. for mortars 1, 2 and 3, respectively.

Tensile bond specimens were cross-brick couplets and modulus of rupture specimens were small brick beams which, quoting from the report: "were built by laying the brick flatwise one upon another with ½-in. mortar joints between to make a beam 8 in. wide by 4 in. deep by 24 in. long. Each was broken twice by testing as a cantilever with load applied at the end; the effective length being 6½ in."

All brick were wet prior to assembling both the tensile bond and modulus of rupture specimens. The specimens were then stored in air where they were wet twice daily with a hose for the first 14 days and thereafter cured dry. Tests were made at the age of 6 months.

Three types of brick were used in both the tensile bond and modulus of rupture test specimens. They are described in the report as follows: "Brand A is a rough textured, wire-cut, stiff mud brick containing considerable coarse

TABLE 2-12

MODULUS OF RUPTURE, BRICK BEAMS

Mortar			Modulus of Rupture③, psi Brick Designation and Absorption④, %		
No.	Mix①	Compressive Strength, psi②	A 12.0	B 13.6	C 8.6
Series 1					
1	1:⅓:4	2930	172	227	342
2	1:½:4½	2470	178	105	165
3	1:1:6	1700	96	115	201
Series 2					
3	1:1:6	1099	227	....	....

① Cement, lime, sand by volume.
② Two-inch by 4-in. cylinders, age 6 months.
③ Small brick beams.
④ Absorption by 48-hr. immersion cold water.

grog; Brand B is made from soft mud using the sand mold process. Brand C is a light buff colored, stiff mud, wire cut face brick." The average compressive strengths for each brand were: Brand A—3035 psi, Brand B—3870 psi, and Brand C—7710 psi.

The suction of the brick used in this investigation are not reported; however, since all brick were wet before assembling the test specimen, it is probable that the suction when laid for all brick was of the order of 10 to 20 grams; also, that the mortar flow was comparable to that reported in Table 2-9 as 125 to 135 per cent.

Effect of Method of Forming. It will be noted in Table 2-12 that the modulus of rupture, as reported for mortar 3, series 2, is approximately 2.4 times the value reported for mortar 3 of the same proportion in series 1. While it is possible that the different limes used in these mortars may account for some of this difference, it is probable that a greater percentage of the difference results from different techniques in assembling the brick beams.

In the tests reported by Pearson in the paper, "Measurement of Bond Between Bricks and Mortar", modulus of rupture specimens were built in which the brick mason followed two techniques. One consisted of tapping the brick down to uniform joint thickness, and in the other hand pressure alone was used to bring the mortar joint to uniform thickness.

Comparing the modulus of rupture values obtained from specimens built by the two methods, it was found that the values obtained where the tapping technique was used exceeded the values from the specimens where hand pressure alone was used in all cases. The author states: "The effect of tapping the bricks is therefore to increase the bond strength from 50 to 100 per cent which suggests that the contact between mortar and brick is much improved by impact or by the vibration resulting from impact."

Effect of Mortar Proportions. As indicated by the data in Table 2-9, for the mortars included in Tables 2-11 and 2-12, there appears to be no consistent relation between strength of mortar and either tensile bond strength or modulus of rupture. However, from the tensile bond strength data, it will be noted in both Tables 2-9 and 2-11 that, for many conditions (mortar flow and suction of brick), the tensile bond strengths obtained with mortar 2 (1 part cement, ½ part lime putty and 4½ parts sand by volume) are higher than the strengths obtained with either mortar 1, containing a lower percentage of lime, or mortar 3 (1 part cement, 1 part lime and 6 parts sand by volume).

While the data are not conclusive, they indicate that there is an optimum ratio of lime to cement for mortars which will develop maximum bond with brick and that, for most conditions, this ratio will lie between the values of 1 and ¼. The character of the lime, as well as the suction of the brick when laid, will undoubtedly affect this ratio; however, for lime putty or highly plastic hydrates (such as pressure hydrated) and for brick suctions not exceeding 20 grams (.7 oz.), the proportions of mortar 2 (1 part cement, ½ part lime, and 4½ parts sand by volume) are recommended.

Effect of Curing. Among the variables studied in the investigation, reported to the Clay Products Institute by Raymond E. Davis, was the effect of different curing conditions on the strength of tensile bond and modulus of rupture specimens. As previously indicated, specimens from which the data included in Tables 2-11 and 2-12 were obtained were stored in air, wet twice daily with a hose for 14 days and thereafter cured dry. Similiar specimens were stored in air and cured dry without any wetting.

Comparing the data obtained from each set of specimens, the report states: "It is instructive to note that on the average the specimens which were cured dry exhibit substantially higher strengths than those which were cured wet. This would indicate that under the conditions of the tests there was sufficient moisture in the mortar and brick when laid to provide for the hydration of the cement without later sprinkling, and that when the specimens were wet cured the additional moisture merely saturated the brick, decreasing the adhesion between brick and mortar as well as the strength of the brick itself. Thus it appears that under normal conditions there is nothing to be gained through wet curing brick or tile masonry after it has been laid. It seems entirely possible that the strengths of the corresponding dry and wet cured groups would not have been substantially different had the corresponding specimens been of the same moisture content at the time of test."

Since all of the specimens on which this conclusion is based were small, they might be expected to dry much more rapidly than 8-in. or 12-in. brick walls, particularly if built of grouted construction. It appears therefore that damp curing of reinforced brick masonry is neither necessary nor desirable.

Effect of Brick Texture. Table 2-13 is compiled from data included in the Bulletin of the Virginia Polytechnic Institute, Engineering Experiment Station Series No. 70, "Mortar Bond Characteristics of Various Brick", by H. R. Forkner, R. S. Hagerman, P. S. Dear and J. W. Whittemore. The brick used in this investigation are described as follows:

A—Stiff mud, wire cut, shale; B—Buff, stiff mud, wire cut, clay; C—Stiff mud, wire cut, clay; D—Stiff mud, wire cut, shale; E—Soft mud, sand molded, clay; F—soft mud, sand molded, clay.

Tensile bond strength was determined from cross-brick couplets and the mortar consisted of 1 part cement, ½ part lime and 4½ parts sand by volume. The lime was a lime putty prepared from a high calcium powdered quicklime. The tensile bond test specimens were stored in air and tested at the age of 28 days.

Surfaces of brick A, B, C and D, designated as R, were the wire-cut bed surfaces, and surfaces designated as S were the smooth die skin surfaces of the faces of the brick. The surfaces designated as R of brick E and F were the struck bed surfaces, and surfaces designated S were the sand finished face surfaces.

TABLE 2-13

EFFECT OF BRICK TEXTURE ON TENSILE BOND STRENGTH OF MORTARS

Brick designation Suction Rate, g	Tensile Bond Strength,③ psi											
	A 10–20		B 5–10		C 10–20		D 5–10		E 50–60		F 50–60	
Surfaces①	R–R	S–S	R–R	S–S	R–R	S–S	R–R	S–S	R–R	S–S	R–R	S–S
Mortar ② Flows												
95–105	34	28	46	28	40	28	34	23	26	19	23	6
115–125	42	39	52	53	58	31	25	31	31	21	38	33
130–140	54	40	65	40	56	34	41	34	37	25	2	11
Average	43	36	54	40	51	31	33	29	31	22	21	17
% S/R	83	....	74	....	61	....	88	....	71	....	81	....

① Surfaces designated R are rough as defined in Section 202(e); those designated S are smooth dye skin or sand finished.
② Mortar proportions: 1 part cement, ½ part lime putty, 4½ parts sand by volume.
③ Cured in air, tested at age 28 days.

All couplets were made of half brick or "soaps", providing an area of mortar in the cross-brick couplet approximately 2 in. square.

The designations in the table, R-R and S-S, mean both surfaces in contact with the mortar joint rough and both surfaces smooth, respectively. Values reported are averages of 2 specimens.

In general, the increase in tensile bond strength with increased flow of mortar and the decrease in bond strength between mortar and brick having suctions exceeding 20 grams, indicated by the data of Table 2-9, are confirmed by these data. Tensile bond strength is less between mortar and smooth brick surfaces than between mortar and roughened surfaces; the reduction averaging about 25 per cent.

Effect of Brick Strength. Failure of brick masonry specimens when subjected to tensile stresses may result from failure of adhesion between brick and mortar, from failure of the mortar or from failure of the brick. For relatively high strength mortars, such as those recommended for use with reinforced brick masonry, and with the great majority of brick as now produced commercially, most failures will be of the first type; that is, failure of adhesion or bond between mortar and brick.

This was the typical failure reported by the authors of the reports from which the tensile bond strengths included in Tables 2-9, 2-11 and 2-13 were obtained. However, Professor Davis reports regarding the data in Table

2-11: "The most common type of failure of the adhesion specimens was in adhesion between mortar and brick, but particularly with the stronger mortars there were numerous failures through cohesion in the brick."

Results of tensile bond tests are reported in Bureau of Standards Research Paper No. 504, "Shear Tests of Reinforced Brick Masonry Beams", by D. E. Parsons, A. H. Stang and J. W. McBurney. Two makes of brick were used in this investigation, both produced from surface clays and formed by the stiff mud, end-cut process. Brick A are described in the report as "irregular in shape, containing lime nodules and considerably laminated", and brick B as "somewhat more regular in shape than brick A, quite free from lime nodules but also laminated." The absorptions by 5-hr. boil were: Brick A—14.7 per cent and brick B—16.1 per cent.

Tensile bond specimens consisted of "two bricks laid flatwise, separated by mortar; the cross-section of the joint being 30 sq. in. for standard size brick." The mortar consisted of 1 part cement, 0.15 parts hydrated lime and 3 parts sand by volume. Two-inch by four-inch mortar cylinders, when stored in water and tested at from 28 to 60 days, had an average compressive strength of 4010 psi. Wetting of brick consisted of total immersion for 1 hr., after which they were allowed to stand in air for ½ hr. before making the test specimens. Dry storage was in air and damp storage was in a damp storage room in which a relative humidity of over 90 per cent was maintained.

Table 2-14, compiled from data included in Research Paper No. 504, shows the results of these tests. It will be noted that specimens, assembled with brick which had been wet prior to laying, developed substantially higher bond strengths than those assembled with dry brick which confirms the conclusions reached by other investigators that tensile bond strength increases with a reduction of the suction of the brick when laid.

TABLE 2-14

RESULTS OF TENSILE TESTS OF BOND BETWEEN MORTAR AND BRICK

Brick		Storage	Number of Specimens	Average Strength psi
Kind	Condition When Laid			
A	Dry	Dry	16	38
		Damp	16	38
	Wet	Dry	20	55
		Damp	20	61
B	Dry	Dry	16	18
		Damp	17	27
	Wet	Dry	20	53
		Damp	20	44

Regarding the data in Table 2-14, the authors state: "Where test specimens represented wetted bricks, the characteristic failure was not at the junction of brick and mortar but was a failure in the brick. Brick A left a 'skin' adhering to the mortar and brick B frequently pulled off or sheared off their flats to a depth of ⅛ in. In other words, when bricks had been wetted, bond between brick and mortar exceeded the strength of the bricks."

Brick of the type used in this investigation represent a very small percentage of the current production of the industry; however, these tests emphasize the importance of a strong ceramic bond between the body of the unit and surfaces which will be in contact with mortar or grout.

Textures, described in Section 202(e) as smooth, scored, combed or roughened, when applied to well burned brick, will develop the maximum bond between mortar and brick. However, if tensile bond strength is a critical factor, the ceramic bond between surface in contact with mortar or grout and body of the brick should be investigated.

(d) Bond—Mortar to Reinforcement. Numerous pull-out tests have been made to determine the resistance to slip of reinforcement when embedded in masonry. These tests consist essentially of determining the force required to cause slip of a reinforcing bar embedded in a block of masonry. Table 2-15 summaries the results of some of these tests.

TABLE 2-15

BOND OF MORTAR TO REINFORCEMENT

Mortar			Bond Strength psi		Reinforcement
No.	Mix①	Compressive Strength, psi			
			(Davis) ②		
			Age 6 mos.	Age 12 mos.	
1 2 3	1:⅓:4 1:½:4½ 1:1:6	2930 2470 1700	480 365 317	456 468 303	⅝″ plain round, embedment 10″, (16 diameters).
			(Converse) ③		
			Av. age 30 da. to 12 wk.		
1 2 3	1:¼:3 1:½:4½ 1:1:6	2330 grout 1300 grout 700 grout	953 788 512		⅜″ deformed round, embedment 3¾″, (10 diameters).
			(Bureau of Standards) ④		
			Age 1 to 2 mos.		
1	1:.15:3	4010(a) 3410(b)	905		½″ deformed square, embedment 8″, (16 diameters).
			(Richart) ⑤		
			Age 28 days		
1 2 3	1:¼:3 1:½:4½ 1:1:6	4100 2995 2095	697 547 450		½″ deformed round embedment 6″, (12 diameters).

① Cement: lime: sand by volume.

(*Richart*) ⑤

1 1	1:¼:3 1:¼:3	4100 1750 grout	Age 28 days 1050(a) 920(b)	⅜″ deformed round, embedment (a) 6″ (16 diameters) (b) 4½″ (12 diameters).

② Report to Clay Products Institute of California by R. E. Davis, 1930. Mortar specimens 2-in. by 4-in. cylinders tested at age of 6 months. Pullout specimens, 8-in. by 8-in. by 10-in. block of brick masonry, no headers. Mortar and pull-out specimens stored in air, wet twice daily with hose for first 14 days, thereafter dry. Author concludes: "It is significant that bond strength is high for the mortars of low lime content but relatively low for those of high lime content. (Results of some high lime mortars not reported in Table 2-15.) It appears that there is little additional strength acquired between the ages of 6 and 12 months."

③ F. J. Converse, report to Brick Manufacturers Association of Southern California, 1941, summarized in paper, "Some Properties of Reinforced Grouted Brick Masonry", by Norman W. Kelch, ASTM Bulletin No. 168, September 1950. Grout "100 per cent flow without shock", test specimens 2-in. by 4-in. cylinders tested at age of 28 days, curing not reported. Pull-out specimens, masonry block 8-in. by 5 in. by 3¾ in. high consisting of 2 brick on edge with 1-in. grout joint between. Author concludes: "Some lime is valuable in grout, the proportions varying with the mixes. For these tests the best mixes were 1:¼:3, 1:½:4½, and 1:1:6."

④ Bureau of Standards Research Paper No. 504, "Shear Tests of Reinforced Brick Masonry Beams," by D. E. Parsons, A. H. Stang, and J. W. McBurney, 1932. Mortar specimens 2-in. by 4-in. cylinders; (a) cured under water, (b) in air; tested at age of 28 to 60 days. Pull-out specimens, brick piers 8 in. by 8 in. in cross-section and 3 brick high. The middle course of brick laid as headers with respect to the top and bottom courses. Dry curing in air, damp curing in damp storage room having relative humidity of over 90 per cent. The authors conclude "The bond strength at an age of from 1 to 2 months as determined by pull-out tests of ½-in. square deformed bars embedded 8 in. in brickwork ranged from 870 to 950 psi. Differences in the kinds of bricks and of curing conditions did not cause significant changes in these bond strengths."

⑤ "Bond Tests Between Steel and Mortar in Reinforced Brick Masonry", by F. E. Richart, published by Structural Clay Products Institute, 1949. Mortar specimens 2-in. cubes tested at age of 28 days, cured in water. Pull-out specimens, brick piers 8 in. by 8 in. in cross-section, various heights, no headers, cured in air. Tests were made on both plain (not reported in Table 2-15) and deformed bars. The author concludes: "The deformed bars used developed definitely higher bond resistance than plain bars, despite the fact that failure with the former generally resulted by splitting of the masonry block, which was weak in lateral tension. For no explainable reason, ⅜-in. deformed bars developed much higher bond stresses than ½-in. bars of the same type."

Effect of Joint Thickness. The report, "Bond Tests Between Steel and Mortar in Reinforced Brick Masonry," by F. E. Richart, also includes the results of tests of pull-out specimens in which the joint where the reinforcement was placed varied in thickness. From the data obtained from these tests, the author concludes: "The studies of relation of joint width to bar size did not show conclusive results, though with ½-in. bars, there was a slight increase in bond resistance as the joint width was increased from ½ to 1 in. This increase was from 10 to 20 per cent. In actual masonry, it would probably be impossible to use the minimum joint widths used in these small pull-out specimens, and a clearance of at least ⅛ in. on each side of the bar seems desirable, especially in vertical joints, and using brick with high suction rates."

Effect of Size of Specimen. Practically all investigators, who have reported the results of pull-out tests of masonry specimens similar to those from which the data included in Table 2-15 were obtained, have noted that failure was often accompanied by splitting of the masonry block and many have suggested that the resistance of walls or beams to splitting should be greater than the resistance of the test specimens.

In order to determine bond strength developed in grouted reinforced brick masonry walls, a test was conducted in 1938 by Raymond G. Osborne, the results of which are summarized in "Some Properties of Grouted Reinforced Brick Masonry", by Norman W. Kelch, ASTM Bulletin No. 168, September 1950.

The following is quoted from the report of this test: "The two test walls under investigation, designated as B and C, were each some 12 ft. 6 in. in length, 3 ft. 0 in. in height and 8 in. in thickness. They were constructed of two parallel brickwork sections enclosing a section of grout approximately 1 in. thick in which were embedded ⅝-in. deformed round steel reinforcement bars spaced 18 in. on centers vertically, and two horizontal bars (below the range of investigation). The brick were of the type known as 'Groutlock', made by the soft mud process, having all around open pore rough surfaces.

"We are informed that the walls were laid on May 11, 1938, and that the construction conformed to the specifications for brick, mortar and grout embodied in the Los Angeles County Building Code, the grout having the proportions of 1:3 and the mortar of 1:3 with addition of 15 per cent lime putty by volume to the cement. A free flowing grout was used (having a slump of some 11.5 in.), poured into and completely filling the center joint at the completion of each course, and trowel-stirred immediately after pouring. The vertical steel (on which all tests were made) was braced to reduce swaying during construction though, throughout the laying of wall C, the bars were violently shaken when each course had been completed.

"Having determined by preliminary work that an embedment in the neighborhood of 5 in. was more than adequate to completely develop the strength of the steel, the test specimens (chosen at random from the vertical steel bars projecting from the top of the walls) were exposed by means of holes cut through the walls and cut off in such fashion as to leave sections of steel embedded between 3.0 in. and 3.68 in. in effective length. On the basis of the theoretical area of 1.18 sq. in. per lin. in. of ⅝-in. smooth round rod assumed, the areas in bond ranged between 3.54 and 4.34 sq. in.

"One of the primary purposes of the test program was to determine the effect of strong agitation on the reinforcement rods during construction (embodied in the construction of wall C). The results of the tests showed no appreciable divergence in trend and, as a result, the data from both walls have been combined in the summary, though the behavior of the individual specimens may be identified as between the two walls by their designations in Table 2-16."

TABLE 2-16

TESTS OF BOND TO STEEL

Slip of Bar, in.	Bond Stress in psi of Embedment					
	B-2	B-6	B-7	C-5	C-6	Average of 5
0.0001	720	625	763	664	775	710
0.0005	955	905	907	766	1170	942
0.0010	1106	1036	1050	901	1250	1068
0.0020	1230	1239	1205	1165	1274	1222
0.0050	1367	1308	1311	1270	1348	1321
0.0100	1425	1365	1374	1286	1420	1373
Ultimate	1750	1550	1842	1605	1775	1704
Note: Approximate yield of steel	1460	1460	1370	1190	1250	....

Bond in Beams. Tests of beams constructed without web reinforcement and designed to fail in bond are reported in "Report of Tests" by Robert R. Schneider, Department of General Engineering, University of Southern California, under date of November 1951. These beams were 5 ft. long and were tested on a span of 55½ in. The beams were 8 in. wide by 18¼ in. high with a center grout joint of 1½ in., and were reinforced with one ½-in. deformed round bar meeting ASTM Specifications A305- for Minimum Requirements for Deformations of Steel Bars for Concrete Reinforcement. Mortar and grout consisted of 1 part cement, 3/10 part lime and 4½ parts sand by volume. The lime was a pressure hydrated hydrate.

Regarding the tests, the report states: "The required 10-diameter bond embedment was obtained in the K beams, and the deformation in the longitudinal steel was measured by means of the SR-4 strain gages. Strain measurements were taken on both the left and right sides of each beam, resulting in an average ultimate bond stress of 464 psi for K-1, 758 psi for K-2 and 740 psi for K-3. Maximum values recorded for each of the three beams were 500 psi, 840 psi and 760 psi, respectively. These results indicate that an eccentric loading condition probably existed; therefore, the maximum value would be the actual bond stress which was developed. Results are shown in Table 2-17. In beam K-2, the lower course of brick separated at the notch before any noticeable slippage of the steel occurred. In the other two specimens, bond between the steel and the grout was destroyed at the ultimate load. No apparent slippage of the steel occurred, however, until the ultimate load was reached."

TABLE 2-17

THEORETICAL AND EXPERIMENTAL BOND STRESSES—K SERIES

Specimen	Reinf. in.	Size in.	Shear v-psi	Experimental Bond Stress				Theo. 1st Crack
				1st Crack		Ultimate		
				Left	Right	Left	Right	
K–1	1½	8¼x16¼	50	204	271	500	428	212
K–2	1½	8¼x16¼	64	223	224	840	675	216
K–3	1½	8¼x16¼	58.5	180	302	720	760	216

From the above data, it appears that bond between steel and masonry depends primarily upon the properties of the reinforcement and the mortar or grout and, assuming sufficient bond between mortar and brick to prevent splitting of the masonry, the properties of the brick have little effect on bond with reinforcement. Mortars having high compressive strengths consistently develop higher bond strengths with reinforcement than lower strength mortars.

(e) Durability. For relatively dry masonry which resists the penetration of excessive moisture and mortars recommended for reinforced brick masonry, the resistance of mortar to alternate freezing and thawing is not a serious problem.

(f) Volume Change. Volume change of mortar due to hardening and cyclic wetting and drying is affected by curing conditions, richness of mix and water content. Mortar hardened in absorbent molds or in clay products masonry shows substantially less volume change (approximately 50 per cent)

than mortar hardened in non-absorbent molds. Mortars composed of cementitious material and aggregate in the proportions of 1:2 or more by volume show relatively slight differences in volume change due to cyclic wetting and drying. Mortars richer in cementitious material than 1:2 show greater volume change and should be used with caution. Shrinkage, particularly during early hardening, increases with increase in water content of the mortar.

Thermal change should be considered in the design of structures; however, the differential thermal volume change between masonry materials does not appear to have an important effect upon the performance of masonry structures.

Results of extensive tests (approximately 82 mixes and 4000 test specimens) of mortars which included the determination of tensile and compressive strengths, workability, linear change, absorption, and the results of freezing and thawing and autoclaving are reported by M. O. Withey and K. F. Wendt in the paper, "Tests of Mortars for Reinforced Brick Masonry", published in Vol. 35, Part II, Proceedings of the American Society for Testing Materials.

It was found in this investigation that dolomitic lime-cement mortars containing over 11 per cent of hydrate by weight of cement "expanded unduly when continuously wet or when moist cured and tested in an autoclave." Also mortars to which calcium chloride had been added in an amount equal to 1½ per cent of the weight of the cement contracted from 50 to 100 per cent more than similar mortars without the addition of calcium chloride.

Volume change, due to unsound ingredients (containing reactive components), may be sufficient to cause disintegration of the Masonry. Examples of this are presented in the paper, "Cracking in Masonry Caused by Expansion in Mortar", by J. W. McBurney, presented at the 1952 Annual Meeting of the American Society for Testing Materials.

In the summary and conclusions, the author states: "It is recommended that masonry cements and cement lime mixtures for use in mortar be required to pass an autoclave test for soundness with a permitted gain in length of not more than 1 per cent."

(g) Grout. Grout is produced by adding sufficient water to a mortar to give it a pouring consistency without causing segregation of the mortar constituents. Since the grout in reinforced brick masonry members performs the dual function of bonding the masonry wythes or tiers together and incasing the reinforcement, relatively high strength mortars are recommended for grout.

Due to its high water content, grout has a high shrinkage as the water is removed by the absorption of the masonry units and, in order to prevent shrinkage cracks from developing, specifications require that the grout be poured as each course of brick is laid and thoroughly puddled.

For grout spaces 2 in. and wider, pea gravel grout is recommended. This grout requires the addition of less water than mortar grouts in order to produce pouring consistency and, consequently, has lower shrinkage.

Compressive strength of grout cubes or cylinders formed in non-absorbent molds are substantially less than the strengths developed by mortars of the same proportions, due primarily to the higher water-cement ratio of the grout. Data reported on compressive strength of grout specimens formed in brick molds indicate that grout cured in masonry develops strengths comparable to that of similar mortars.

(h) Admixtures. During recent years, a wide variety of admixtures for masonry mortars has been placed on the market. Most of these products are proprietary and their composition is not disclosed by the manufacturer. In general, however, they classify as grinding aids and air-entraining agents, water repellents, or wetting and air-entraining agents.

Data on the effect of these admixtures on the properties of mortar, as a rule, are limited to their effects on flow, water retentivity and strength, and little information has been published as to their effect on bond, either between mortar and brick or between mortar and reinforcement.

In the series of bond tests between steel and mortar, conducted at the University of Illinois in 1942 and reported by Professor F. E. Richart in the bulletin, "Bond Tests Between Steel and Mortar in Reinforced Brick Masonry", published by the Structural Clay Products Institute, pull-out tests of three groups of specimens were made with mortar mix A (1 cement; ¼ lime; 3 sand by volume) to which had been added the following admixtures: Vinsol resin—0.04 per cent by weight of cement ground into the cement; calcium stearate—ammonium stearate was used in sufficient quantity to provide for the formation of 0.2 per cent of calcium stearate by weight of the combined cement and lime in the mortar; Orvus, a wetting agent and air-entraining material manufactured by Proctor and Gamble Distributing Company, Cincinnati, Ohio, in the amount of 0.04 per cent by weight of the combined cement and lime in the mortar.

Regarding these tests, the author states: "Comparing the maximum bond resistance of the several mortars, the mortar containing calcium stearate and the grout produced bond values equal to the basic mortar A (1 cement: ¼ lime: 3 sand by volume); mortar F, identical with A, except that a different lot of cement was used, gave slightly higher values, and all others were consistently lower. The mortars containing Vinsol resin and Orvus gave roughly 20 per cent lower bond strength than the basic mixture."

There is some evidence, based largely upon field experience, that certain admixtures have a detrimental effect upon bond between mortar and brick. For these reasons, admixtures are not recommended for use in mortars for reinforced brick masonry unless it has been established by experience or laboratory tests that they will not materially impair the mortar bond.

(i) Summary. The following summarizes the conclusions which may be drawn from the data presented in Section 203, Mortar.

Workability and Water Rententivity. Workability is affected by properties of the ingredients, particularly the lime. Improved workability is obtained in mortars containing highly plastic limes and well graded sand. Numerous admixtures are available which improve mortar workability; however, unless the effect of these admixtures on bond between mortar and brick and between mortar and reinforcement is known, they should be used with caution.

Water retentivity of mortar increases with increased proportions of lime; however, the properties of the lime have an equal or greater effect than the proportion. Highly plastic lime putties produce mortars of high water retentivity.

Strength. Strength of mortar, as measured by the compressive strength of standard cubes or cylinders or by the tensile strength of standard briquets, increases with increased proportions of cement and with decreased water-cement ratio. High strength mortars have higher bond strength between mortar and reinforcement than low strength mortars. However, for mortars

recommended for use in reinforced brick masonry, there does not appear to be a consistent relationship between mortar strength and strength of bond between mortar and brick.

Mortar strength may be used as a check on proportion and mixing of mortars and for hydraulic mortars is a reasonably good measure of resistance to freezing and thawing. Mortar affects compressive strengths of masonry walls approximately proportional to the cube root of its strength.

Bond—Mortar to Brick. Tensile bond strength between mortar and brick increases with increased flow of the mortar. In masonry construction mortar should be mixed with the maximum amount of water consistent with workability. Tensile bond strength decreases rapidly as brick suction increases above 20 grams (0.7 oz.). In masonry construction all brick having suctions exeeding this amount should be wet before laying.

Bond—Mortar to Reinforcement. Bond strengths of from 500 to 1000 psi can be obtained with A-1 and A-2 mortars; 1 part cement, ¼ part lime and 3 parts sand, and 1 part cement, ½ part lime and 4½ parts sand by volume, respectively.

Durability. Mortars recommended for use in reinforced brick masonry have high resistance to freezing and thawing, and in reinforced masonry which resists the excessive penetration of moisture, durability of mortar is not a problem.

Volume Change. Thermal volume change should be considered in the design of structures. Mortars containing reactive chemical compounds, such as unhydrated magnesium oxide, may expand sufficiently to cause disintegration of the masonry.

Admixtures. Admixtures are, as a rule, added to mortars to produce water repellency, to improve workability or to increase water retentivity. Certain admixtures have a deleterious effect upon bond between mortar and brick and between mortar and reinforcement. They should not be used in mortars for use in reinforced brick masonry unless laboratory data or performance records are available which indicate that such mortar will develop satisfactory bond.

204. REINFORCEMENT

The reinforcement recommended for principal reinforced brick masonry members is steel deformed bars meeting the requirements of ASTM Tentative Specifications A305-. Comparative bond tests of this type of reinforcement and plain and "old type" deformed bars are reported in the paper, "Bond of Concrete Reinforcing Bars", by Arthur P. Clark, published in the 1949 Journal of the American Concrete Institute. Data included in this paper indicate that bond stresses developed in both beams and pull-out specimens by bars meeting the ASTM requirements were from 50 to 100 per cent greater than with plain or old type bars.

As a result of these tests and others, the American Concrete Institute revised its building code requirements for reinforced concrete in 1951 by increasing the allowable unit stresses in bond by 75 per cent for deformed bars meeting the ASTM requirements.

Few data are available at this time (1953) on the bond developed by ASTM A305- bars in brick masonry; however, it seems reasonable to assume the

bond will be substantially increased over that obtained with the old type bars and that allowable bond stresses may be increased accordingly.

205. UNREINFORCED BRICK MASONRY

While the performance of reinforced brick masonry structures differs materially from unreinforced brick masonry due to the effect of the reinforcement, many factors which affect the performance of the latter have a similar effect upon reinforced brick masonry. Data will be presented in this section on compressive, transverse and shearing strengths and modulus of elasticity of unreinforced brick masonry.

(a) Compressive Strength. For many years tests have been conducted on brick masonry walls and piers to determine the compressive strength of the construction. Data, obtained during the period 1882 to 1924 from tests of 708 specimens, were compiled by the Building Code Committee of the U. S. Department of Commerce under date of March 1926 and given wide distribution. These data, together with the results of subsequent tests, indicate that there are a number of factors affecting the compressive strengh of brick masonry walls and piers.

In the paper, "The Effect of Strength of Brick on Compressive Strength of Masonry", by J. W. McBurney, published in Vol. 28, Part II, Proceedings of the American Society for Testing Materials, the following are listed as the principal factors affecting the compressive strength of brick masonry: (1) workmanship, (2) type of construction (design of walls, hollow or solid, number of headers, etc.), (3) strength of mortar, (4) adhesion of brick to mortar, (5) uniformity and regularity of size and shape of brick units.

Attempts have been made to evaluate the effect of these factors upon the compressive strength of brick masonry, and from the data available it is possible to predict compressive strength of masonry within reasonable limits for known materials and a known and uniform quality of workmanship. However, because of the many variables which must be evaluated in such an estimate, the compressive strengths of wallettes or piers, 2 to 3 ft. high, built with similar materials, the same type of workmanship and of the same design as the proposed construction, have been found to be more accurate measures of the compressive strengths of full size members.

In 1937 ASTM Committee C-15 appointed a working subcommittee on tests for brick prisms to determine a basis for predicting the compressive strength of brick masonry walls from the strength of relatively small masonry test specimens. This committee formulated a program of tests which were carried out at Columbia University under the direction of Professor W. J. Krefeld and were reported by him in a paper, "Effect of Shape of Specimen on the Apparent Compressive Strength of Brick Masonry", which is appended to the 1938 report of Committee C-15, ASTM Proceedings, Vol. 38, Part I.

The brick used for construction of the specimens were soft mud, sand struck clay brick having an average compressive strength of 5124 psi, an average absorption by 5-hr. boil of 19.1 per cent and a suction when laid of 39.7. It was found that the compressive strength of the specimens decreased at a fairly uniform rate with increase of ratio of height to thickness (h/d) to 6, beyond which the decrease up to an h/d ratio of 12 was slight. As a result of these tests, the author proposes the correction factors given in Table 2-18.

TABLE 2-18

STRENGTH CORRECTION FACTORS FOR BRICK MASONRY PRISMS

Ratio, Height to Thickness h/d	Strength Correction Factor	Ratio, Height to Thickness h/d	Strength Correction Factor
1.1	0.45	4.5	0.93
1.5	0.59	5.0	0.96
2.0	0.67	5.5	0.98
2.5	0.75	6.0	1.00
3.0	0.80	8.0	1.03
3.5	0.85	10.0	1.06
4.0	0.89	12.0	1.09

The factors of Table 2-18 are based on a height to thickness ratio of 6 and are to be applied to test specimens (prisms) built of similar materials, with the same method of bonding and by the same workmanship that will be used in the full size member. These factors may also be used to estimate the strength of reinforced brick masonry walls and piers with minimum reinforcement. However, it has been found that the ultimate compressive strength developed in reinforced brick masonry flexural members as determined by the straight line formula

$$f_m = \frac{2M}{jkbd^2}$$

is more nearly equal to the strength of masonry prisms having an h/d ratio of 2 and built of similar materials, with the same bonding arrangement and similar workmanship as the flexural member. This relationship will be discussed in Section 206(b).

Wall Strength. Results of compressive tests of 168 solid and hollow brick walls each 6 ft. long and about 9 ft. high and of 129 wallettes about 18 in. long and 34 in. high are reported in National Bureau of Standards Research Paper No. 108, "Compressive Strength of Clay Brick Walls", by A. H. Stang, D. E. Parsons and J. W. McBurney. Ten types of walls were tested, including 8-in. and 12-in. solid brick walls, 3 types of 8-in. rowlock and 4 types of 12-in. rowlock walls, and 4-in. economy walls.

The purposes of these tests, as summarized in the report were:

1. To determine the relation between the several physical properties of the brick and the strength of walls built therefrom.

2. To obtain the comparative strengths of different types of solid and of hollow walls of brick.

3. To determine the effect of damp curing on the compressive strength of brick walls.

4. To determine the relation between the strength of the mortar and the strength of the wall.

5. To determine the effect of the quality of workmanship on wall strength.

6. To compare the compressive strengths of masonry specimens of different sizes.

Brick from 4 different sources were used in this investigation, designated in the report by the name of the region in which they were produced. They are described as follows:

Chicago brick were made from surface clay and formed by the end-cut, double-column, stiff-mud process. They are rather irregular in shape and contain lime nodules.

Detroit brick were formed by the soft-mud process from surface clay. Like many soft-mud brick, they were formed with a frog or depression in one face approximately 0.4 in. deep.

Mississippi brick were surface clay brick formed by the dry press process. Regularity of size and shape was the outstanding characteristic of these specimens.

New England brick were formed by the soft-mud process from surface clay and were "sand struck". The specimens contained a shallow frog or depression, were very hard burned, and rather irregular in shape and size.

The physical properties of these brick are given in Table 2-19. Each value reported is the average of 50 tests, except shearing strength which is the average of 4 tests for New England brick and the average of 10 tests for all others.

TABLE 2-19

AVERAGE PHYSICAL PROPERTIES OF THE BRICK

Kind of Brick	Absorption		Modulus of Rupture		Compressive Strength		Tensile Strength,	Shearing Strength,
	5 hr. boil, per cent	48 hr. cold, per cent	Flat-wise, psi	Edge-wise, psi	Flat-wise, psi	Edge-wise, psi	psi	psi
1. Chicago	16.5	11.7	1225	1340	3280	3350	417	1100
2. Detroit	22.3	20.7	670	680	3540	3270	222	1165
3. Mississippi	21.7	16.7	820	760	3410	3625	317	1590
4. New England	9.2	6.9	1550	1640	8600	11470	601	3550

Two mortars were used in the investigation: A cement mortar consisting of 1 part portland cement: 0.10 parts lime: 3 parts sand by volume; and a cement-lime mortar consisting of 1 part portland cement: 1 part hydrated lime: 6 parts sand by volume. Compressive strengths of these mortars are given in Table 2-20.

TABLE 2-20

COMPRESSIVE STRENGTH OF MORTAR SPECIMENS

Mortar	Proportions ① (by volume)	Strength ②	
		Cured wet, psi	Cured, dry, psi
C–L Cement-lime	1C:1L:6S	1100	750
C Cement	1C:1/10L:3S	3260	1950

① C = cement, L = lime, S = sand.
② Cylinders 2 in. in diameter, 4 in. long, tested at 57 to 62 days.

Two types of workmanship were used in building the test specimens. Type A, designated as "inspected", was characterized by complete filling of all mortar joints and smooth (unfurrowed) bed joints. Type B, designated as

TABLE 2-21

COMPRESSIVE STRENGTH OF SOLID BRICK WALLS

Wall No.	Brick	Mortar	Average Strength, psi		Ratio Wall to Wallette
			Walls	Wallettes	
8-in. Thickness; Type B, Commercial Workmanship					
7-8-9	1	C-L	595	715	.83
13-14-15	1	C	665	790	.84
8-in. Thickness; Type A, Inspected Workmanship					
19-20-21	2	C-L	910	1045	.87
34-35-36	3	C-L	1160	1410	.82
94-95-96	4	C-L	1790	2300	.78
22-23-24	2	C	1080	1350	.80
37-38-39	3	C	1380	1630	.85
97-98-99	4	C	2635	3700	.71
12-in. Thickness; Type B, Commercial Workmanship					
10-11-12	1	C-L	580	655	.89
16-17	1	C	655	755	.87
12-in. Thickness; Type A, Inspected Workmanship					
25-26-27	2	C-L	985	1020	.97
40-41-42	3	C-L	1300	1275	1.02①
100-101-102	4	C-L	1890	2460	.77
18	1	C	1165	1190	.98
28-29-30	2	C	1210	1335	.91
31-32②	2	C	1105	1160	.95
43-44-45	3	C	1640	1760	.93
46-47②	3	C	1590	1865	.85
48	3	C	1465	1640	.89
103-104-105	4	C	2790	3360	.83
106-107②	4	C	2510	3075	.82
108	4	C	2720	3200	.85

① Not included in averaging ratios.
② Damp cured.

"commercial", was characterized by deeply furrowed bed joints, partially filled head joints, and practically no mortar in the longitudinal vertical (collar) joints.

Bids were obtained for the construction of Type B walls from a number of brick masons and the work was awarded to the lowest bidder who received no instructions as to the type of workmanship which should be employed in building the specimens, other than it should be the same as would be used in commercial work.

The 8-in. and 12-in. solid walls were laid in common American bond with headers every sixth course.

The average compressive strengths of the solid brick walls and wallettes included in this investigation are shown in Table 2-21, also the ratio of wall strength to similar wallette strength. Based on the factors included in Table 2-18, this ratio might be expected to be .82 for the 8-in. walls and .74 for the 12-in. walls.

It will be noted that the actual ratio of wall strength to wallette strength for the 8-in. walls (average value .81) is very close to the estimated, but the value for the 12-in. walls (average value .89) is substantially higher. This variation may be due in part to the difference between the properties of the materials used in constructing the walls and wallettes and the materials used for building the test specimens on which the correction factors in Table 2-18 are based. However, the report indicates that the type of workmanship used in building the wallettes of Type A workmanship was inferior to that used for the construction of the walls.

Quoting from the report, "The mason who built the specimens of series 2 did not seem to like to build the small walls and on several occasions had to be told to use as much care on them as he used on the laboratory specimens. * * * In drawing conclusions from the tests of series 2 (Type A workmanship), it is therefore to be remembered that relatively large variations have occurred in many of the groups."

As will be indicated later, Type B, commercial workmanship, produced walls having compressive strengths approximately 60 per cent of similar walls in which Type A workmanship was used. If it can be assumed that the workmanship employed on the wallettes representing the 12-in. walls reduced their strength to 80 per cent of that obtainable with the same type of workmanship used in building the walls, the corrected coefficient (.71) would approximate that obtained from Table 2-18.

Data included in Table 2-21 are summarized in Table 2-22.

TABLE 2-22

AVERAGE COMPRESSIVE STRENGTH OF SOLID BRICK WALLS

Kind of Brick	Compressive strength of half brick, flatwise psi	Average Compressive Strength Solid Brick Walls ①			
		Inspected Workmanship		Commercial Workmanship	
		Cement-lime Mortar, psi	Cement Mortar, psi	Cement-lime Mortar, psi	Cement Mortar, psi
Chicago	3280	....	895	585	660
Detroit	3540	945	1145	....	....
Mississippi	3410	1300	1550	....	870
New England	8600	1875	2855	....	2030

① Tested at 57 to 62 days.

These data indicate the effect of strength of brick, strength of mortar and type of workmanship on the compressive strength of solid brick walls. Table 2-23 is compiled from Bureau of Standards Report BMS5 and shows a similar relation between brick strength, mortar strength, type of workmanship and compressive strength of solid brick walls.

TABLE 2-23

COMPRESSIVE STRENGTH OF SOLID BRICK WALLS

Wall Desig.	Compressive Strength psi		Workmanship② Type	Wall Strength Compressive kips/lin. ft.	Estimated Max. Compressive Stress, psi
	Brick	Mortar①			
AA	17600	C 3220	A	324.0	6000
AB	2670	C–L 640	B	60.5	1130
AC	2670	C–L 640	A	101.0	1850

① Mortar: C 1 cement: ¼ hydrated lime: 3 sand by volume.
C–L 1 cement: 1 hydrated lime: 6 sand by volume.
Strength of 2-in. cubes at 28 days cured in water.
② Workmanship as defined for Table 2-21: A—Inspected; B—Commercial.

Walls, from which the data included in Table 2-23 were obtained, were 8-in. thick, 4 ft. long and approximately 8 ft. high, laid in common bond with headers every sixth course. Compressive load was applied at 1/3 the thickness of the specimen from the inside face. The maximum compressive stress for this loading, as computed from the formula

$$f = \frac{P}{A} \pm \frac{Pe}{S}$$

where f = compressive stress, pounds per square inch
P = total applied load, pounds
A = area of wall, square inches
e = eccentricity, inches
S = section modulus of the wall section, inch ^{3}

is twice the average compressive stress (P/A). However, deflections of the two wall faces observed during the test indicate that the stress was not distributed in accordance with the assumption upon which the above formula is based, due partly to the fact that the walls were capped with a rigid steel plate.

Based on the deflections of the opposite faces, it appears that the compressive stress in the extreme fibers of the face nearest the eccentric load was from 75 to 85 per cent greater than the average stress, instead of 100 per cent greater. The estimated maximum compressive stress, shown in Table 2-23, is approximately 180 per cent of the average compressive stress.

As previously stated, due to the many factors which affect the strength of solid masonry walls, wall strength can be predicted most accurately from the strengths of masonry prisms, constructed of similar materials, the same workmanship and with the same bonding arrangement that will be used in the construction to which the correction factors in Table 2-18 may be applied. However, if data are not available on the strength of masonry prisms, brick masonry wall strengths may be predicted with reasonable accuracy by the following method:

For brick walls laid up with mortar having a 2-in. cube compressive strength of not less than 2500 psi when cured wet for 28 days and with Type A "inspected" workmanship, wall strength will be approximately one-third of the compressive strength of brick, provided the brick strength exceeds 4500 psi. For brick having compressive strengths of 2500 psi, the wall strength will be approximately 50 per cent of the strength of the brick.

For brick strengths between 2500 and 4500 psi, the estimated wall strengths should be proportioned between 50 and 33 per cent of brick strength.

All other factors remaining constant, brick wall strength varies approximately as the cube root of mortar strength. Brick walls laid up with cement-lime mortars (1:1:6) having 2-in. cube compressive strengths of from 600 to 1200 psi when cured wet for 28 days, and with Type A, "inspected", workmanship, may be expected to have strengths of 60 to 70 per cent of the strength of similar walls built with cement mortar (1:1/4:3).

All other factors remaining constant, walls constructed with Type B, "commercial", workmanship will have strengths approximately 60 per cent of similar walls constructed with Type A, "inspected", workmanship.

Effect of Curing. Regarding the effect of damp curing on the compressive strength of brick masonry walls, the authors of Research Paper 108 conclude:

"Walls laid in cement mortar and kept damp for 7 days after construction were not stronger at the age of 60 days than similar walls cured in the laboratory under ordinary conditions."

Effect of Mortar Strength. The effect of mortar strength on wall strength is summarized in the conclusions of Research Paper 108 as follows:

"The strength of the solid walls, which were built by contract, varied about as the cube root of the compressive strength of the mortar cylinders (2 in. in diameter and 4 in. long) which were made from the mortar of the walls and cured under the same conditions. For the solid walls, built under careful supervision, the increase in strength for cement-mortar walls over those laid in cement-lime mortar was about 20 per cent for walls of Detroit and Mississippi brick and about 50 per cent for walls of New England brick, while the average ratio of the cube roots of the mortar cylinder strengths (cement and cement-lime mortars) was 1.38."

(b) Transverse Strength. The resistance of brick masonry walls to lateral forces applied normal to the face of the wall depends primarily upon the strength of the bed joints through which failures normally occur. For well burned brick and relatively high strength mortars, such as those recommended for use in reinforced brick masonry, failure will usually be of the adhesion or bond between mortar and brick.

As indicated in Section 203(c), bond between mortar and brick is materially affected by mortar composition and flow, suction of brick when laid and the technique of forming the mortar joint. Obviously, incompletely filled joints, due to furrowing or partial filling, develop much less strength than completely filled joints.

Highest bond strength is associated with relatively high strength mortars having maximum water content consistent with workability, with brick suction when laid of .7 oz. or under, with completely filled (unfurrowed) mortar joints, and the application of pressure to the unit during the forming of the joint.

While data on the transverse strength of brick masonry walls are not as extensive as those available on the compressive strength of walls and piers, transverse tests of specimens, constructed of materials representative of those used in construction, which have been conducted in recent years provide data from which it is possible to estimate the expected transverse strength of brick masonry walls if the type of workmanship and properties of the materials entering into the construction are known.

Standard methods of conducting transverse tests on masonry walls are included in ASTM Tentative Methods of Conducting Strength Tests of Panels for Building Construction, E72-. This standard provides that the test specimens shall have a height of 8 ft. and a nominal width of 4 ft. and that they shall be tested in a vertical position on a span of 7 ft. 6 in. by applying two transverse loads at the quarter points of the span.

Fig. 2-5
Typical Transverse Test Assembly

Fig. 2-5 shows a typical test assembly. The test panel shown was constructed of "SCR brick"*, a unit developed by the Structural Clay Products Research Foundation, whose dimensions are given in Fig. 2-6. Tests of "SCR brick" walls were conducted by Armour Research Foundation of the Illinois Institute of Technology for the Structural Clay Products Research Foundation in 1951. Specimens were constructed with each of 3 mortar mixes.

* Reg. U. S. Pat. Off. Pat. Pend., SCPRF.

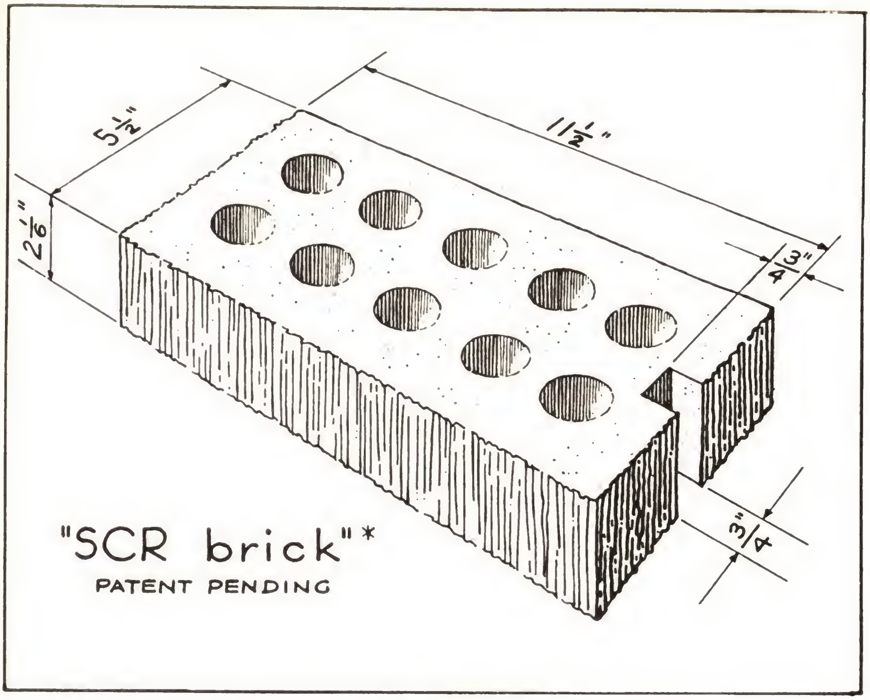


Fig. 2-6
Dimensions of Typical "SCR brick"

Regarding the workmanship used in constructing the specimens, the report states: "All joints were completely filled with mortar and flush with the faces of the wall. There was some mortar penetration into all the cores of the brick. The degree of penetration varied from almost a full core to about ¼ in. of penetration. The instructions to the mason were to place enough mortar on the bed joint to provide an adequate core key when the mortar was spread without furrowing. Inspection of the walls after testing showed that an effective key had been produced in the majority of the cores."

Walls were tested at ages of from 34 to 43 days, cured in air.

Physical properties of the "SCR brick" are given in Table 2-24 and the transverse strength of the walls is shown in Table 2-25. The values reported are the average of 3 specimens.

TABLE 2-24

PHYSICAL PROPERTIES OF "SCR brick"

Compressive Strength psi	Modulus of Rupture psi	Weight, Dry lb.	Water Absorption				
			24-hr. Cold %	5-hr. Boil %	Saturation Coefficient C/B	1 min. Partial Immersion① Dry	1 min. Partial Immersion① As Laid
11140	458	8.43	8.8	11.1	0.79	14.1	13.7

① Absorption is grams per 30 sq. in. immersed on flat side in ⅛ in. of water.

TABLE 2-25

TRANSVERSE STRENGTH OF "SCR brick" WALLS

Mortar		Wall Strength Max. Equivalent Uniform Load psf	Modulus of Rupture psi
Mix	Compressive Strength① psi		
1:¼:3	4590	32.8	45.5
1:½:4½	2745	62.4	86.4
1:1:6	1695	26.1	36.3

① Two-inch cubes stored in water, tested at 28 days.

Modulus of rupture, listed in Table 2-25, is computed from the formula:

$$f = M/S$$

where f is modulus of rupture, psi; M is bending moment, inch pounds; and S is section modulus, inch ^{3}. The bending moment was computed from the formula:

$$M = \frac{WL}{8}$$

where M equals bending moment, inch pounds; W is total load, pounds; and L is span, inches.

As will be noted from Table 2-25, the modulus of rupture of the walls, built with the mortar composed of 1 part cement, ½ part lime and 4½ parts sand by volume, is substantially higher than the value obtained from specimens constructed with either of the other two mortars. While it is possible that factors other than mortar composition affected the strength of specimens built with this mortar, the indication that the optimum ratio of cement to lime for maximum bond strength lies between ¼ and 1, which was discussed in Section 203, is also apparent from these tests.

Transverse strength of brick masonry walls are reported in National Bureau of Standards Report, BMS5, "Structural Properties of Six Masonry Wall Constructions", issued in November 1938. Tables 2-26 and 2-27 give the physical properties of the brick and mortar used for the construction of these walls, and Table 2-28 summarizes the results of the transverse tests. Values shown are the average of 3 tests. Specimens were cured in air and tested at the age of 28 days.

Test specimens AA were built with high strength brick laid in common American bond with headers every sixth course and cement mortar. All joints were completely filled with mortar. The bed joints were flat. The head and collar joints were filled by buttering heavily the ends of brick laid in the facing and both the ends and sides of brick laid in the backing. The mortar was applied to the brick by scraping the trowel against the lower edges and unfilled portions of the joints were filled by slushing mortar from above. The joints were cut flush with the face of the specimens. This is the type of workmanship characterized as Type A in Section 205(a).

Test specimens AB were built with medium strength brick laid in common American bond with headers every sixth course and cement-lime mortar. The

TABLE 2-26

PHYSICAL PROPERTIES OF BRICK

Brick	Compressive Strength, psi	Modulus of Rupture, psi	Water Absorption					Weight, dry lb.
			24-hr. cold, C per cent	5-hr. boil, B per cent	Ratio C/B	1-min. Partial Immersion ①		
						Dry	As laid	
High-strength	17,600	2,275	1.9	3.45	0.53	8	8	5.85
Medium-strength	2,670	550	11.3	15.1	0.74	23	11	4.49

① Immersed on flat side in 1/8 in. of water. Absorption in grams per 30 sq. in.

TABLE 2-27

PHYSICAL PROPERTIES OF MORTAR

Kind of Mortar	Proportion, by Volume	Water Content, by Weight of Dry Materials, per cent	Flow, per cent	Compressive strength	
				Air Storage, psi	Water Storage psi
Cement	1C:0.25L:3S	19.6	113	1390	3220
Cement-lime	1C:1L:6S	23.3	107	440	640

C = cement, L = lime and S = sand.

joints were not completely filled with mortar. The bed joints were furrowed; collar joints were left open, and only the outside of the head joints was filled by lightly buttering the outer edges of the brick. The joints were cut flush with the face of the specimens. This is the type of workmanship characterized as Type B in Section 205(a).

Test specimens AC were built with medium strength brick laid in common American bond with headers every sixth course and cement-lime mortar. The workmanship was Type A, the same as used for test specimens AA.

TABLE 2-28

TRANSVERSE TESTS OF BRICK WALLS

Wall Type	Equivalent Uniform Load, psf				Modulus of Rupture ①, psi			
	1	2	3	Average	1	2	3	Average
AA	115	120	140	125	73.6	76.7	89.5	79.9
AB	53.3	38.0	52.3	48	34.7	24.7	34.0	31.1
AC	85	80	82	82	53.6	50.4	51.7	51.9

① Tested at age of 28 days.

While the data from these tests are not sufficient to warrant generalizations, it is interesting to note that the average transverse strength of walls AC is approximately 70 per cent of the strength of walls AA where the prin-

cipal variable affecting transverse strength is the strength of the mortar. This is the same percentage suggested in Section 205(a) for estimating the difference in compressive strength of walls similar in all respects except the mortar.

In like manner, it will be noted that the average transverse strength of walls AB is approximately 60 per cent of the strength of walls AC which is the same percentage suggested for estimating the difference in compressive strength of walls constructed of Type A and Type B workmanship.

(c) Shear. The racking test, as described in ASTM Tentative Methods of Conducting Strength Tests of Panels for Building Construction, E72-, is usually considered a measure of the shearing strength of brick masonry walls. The specimen used for this test is a wall 8 ft. high by 8 ft. long which is tested by restraining it from sliding or rotating and applying a horizontal force near the upper end of the specimen as indicated in Fig. 2-7.

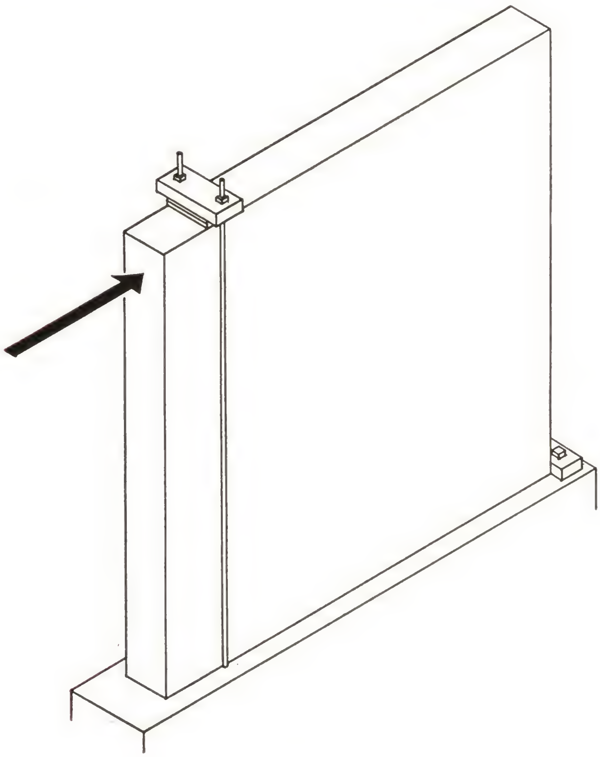


Fig. 2-7
Racking Load on Wall Specimens

Fig. 2-8 shows the assembly used at Armour Research Foundation of Illinois Institute of Technology in conducting the racking test on an "SCR brick" wall.

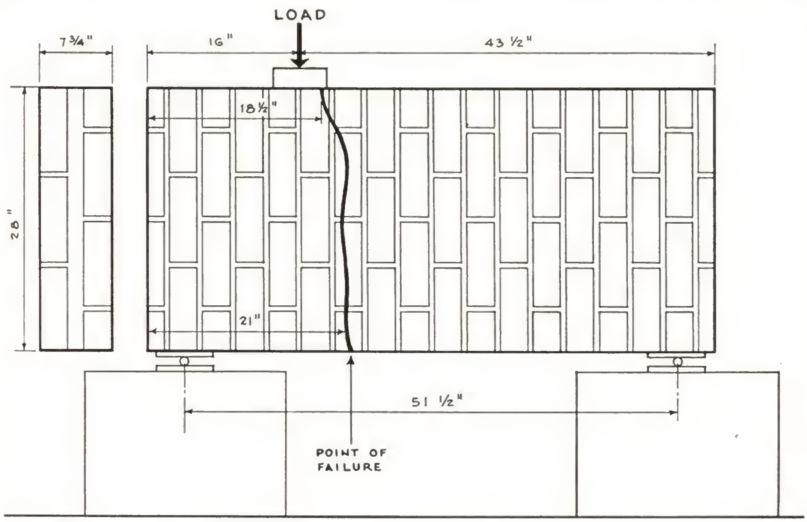
Failure of walls in the racking test is usually in diagonal tension and the plane of failure extends from near the top corner, where the load is applied, diagonally downward toward the bottom support.

Fig. 2-8
Typical Racking Test Assembly

Bureau of Standards Report BMS5 includes results of racking tests of walls similar (materials and workmanship) to those described in Section 205(b) which were subjected to the transverse test. However, none of these walls failed under a total load of 50,000 lb. (6.25 kips per lin. ft. of wall) which was the maximum capacity of the testing apparatus.

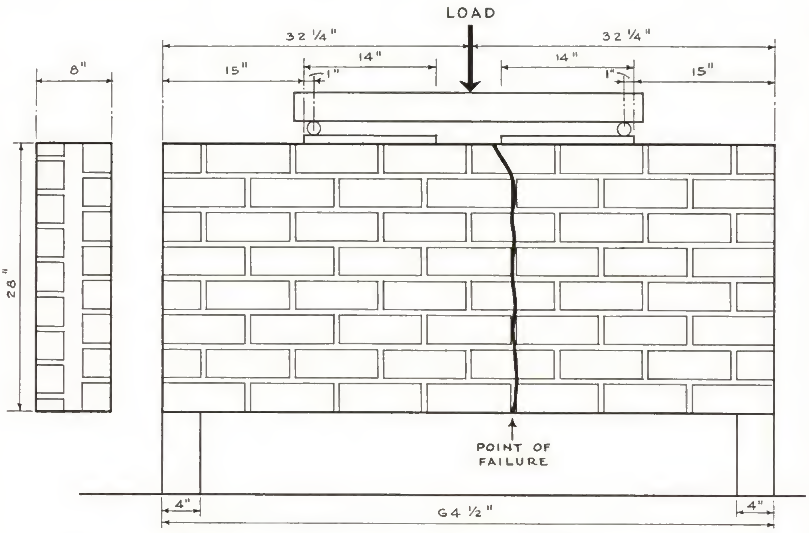
Assuming shearing strength as equal to the total horizontal force divided by the cross-section of the wall, all of these walls, including wall AB (1 cement: 1 lime: 6 sand by volume mortar, and commercial workmanship), developed shearing strengths exceeding 64 psi.

Racking tests were made on "SCR brick" walls, constructed with a mortar consisting of 1 part cement, 1 part lime and 6 parts sand by volume, and similar units and workmanship to those used in the construction of the trans-



GROUTLOCK BRICK LAID UP WITH GROUT ONLY. GRAVITY GROUTED WITH SLIP FORMS. OFFSET JOINTS HORIZONTALLY, ONE SIDE OF COLUMN STARTED AND FINISHED WITH HALF BRICKS.

(A)



GROUTLOCK BRICK LAID UP IN THE REGULAR MANNER. OFFSET JOINTS, ONE SIDE OF BEAM STARTED AND FINISHED WITH HALF BRICKS.

(B)

Fig. 2-9
Wall Sections Tested as Short Beams

verse test specimens described in Section 205(b). The average shearing strength of 3 specimens, computed as previously indicated, was 124 psi.

Tests of wall sections as short span (approximately 5-ft.) beams were conducted by the Smith-Emery Company of Los Angeles, California, to determine shearing strength of unreinforced grouted brick masonry walls. Results of these tests are summarized in the paper, "Some Properties of Grouted Reinforced Brick Masonry", by Norman W. Kelch, ASTM Bulletin 168, September 1950. Fig. 2-9 illustrates the type of wall sections tested.

In Fig. 2-9 (A) the applied load is comparable to a lateral force applied parallel to the wall face, and in Fig. 2-9(B) the applied load simulates a vertical force applied to an unsupported wall section.

Mortar used for the construction of these specimens consisted of 1 part cement, ¼ part lime putty and 3 parts sand by volume, and grout consisted of 1 part cement, .15 part lime putty and 3 parts sand by volume.

Test specimen (A) was loaded with a concentrated load 15 in. from one support and specimen (B) with 2 equal concentrated loads each 15 in. from the adjacent supports. Both specimens failed in tension and ultimate shearing stresses were not obtained.

Table 2-29 gives modulus of rupture and maximum shear developed at ultimate load, computed by dividing the moment at the point of failure by the section modulus and by dividing the maximum shear by the area of the cross-section, respectively.

TABLE 2-29

TENSILE AND SHEARING STRENGTHS OF MASONRY

Beam Designation	Modulus of Rupture, psi	Shearing Stress, psi
2–9(A)	161	71.5
2–9(B)	185	61.5

(d) Modulus of Elasticity. In the tests reported in Bureau of Standards Research Paper 108, described in Section 205(a), compression deflections were observed and stress strain curves were plotted for most of the walls tested.

Regarding the modulus of elasticity, the report states: "For the walls laid in cement-lime and cement mortars, the stress strain curves are, for low stresses, approximately straight lines. In general, however, there is no value for modulus of elasticity which is consistent over a large stress range."

Values of modulus of elasticity, obtained by dividing the stress by the corresponding compressive strain, are reported for stress ranges of 0 to 200 psi for walls built with cement-lime mortar and stress ranges of 0 to 250 psi for cement mortar. These values of modulus of elasticity vary from 2,652,000 psi to a low of 473,000 psi and have been plotted in Fig. 2-10 as ordinates against corresponding ultimate wall strengths, listed in Table 2-21, as abscissae.

It will be noted from Fig. 2-10 that there is no consistent relationship between ultimate wall strength and modulus of elasticity. However, most values lie between line A and line B of Fig. 2-10 whose equations are $E = 1200 f'$ and $E = 700 f'$, respectively, where E is modulus of elasticity and f' is ultimate wall strength.

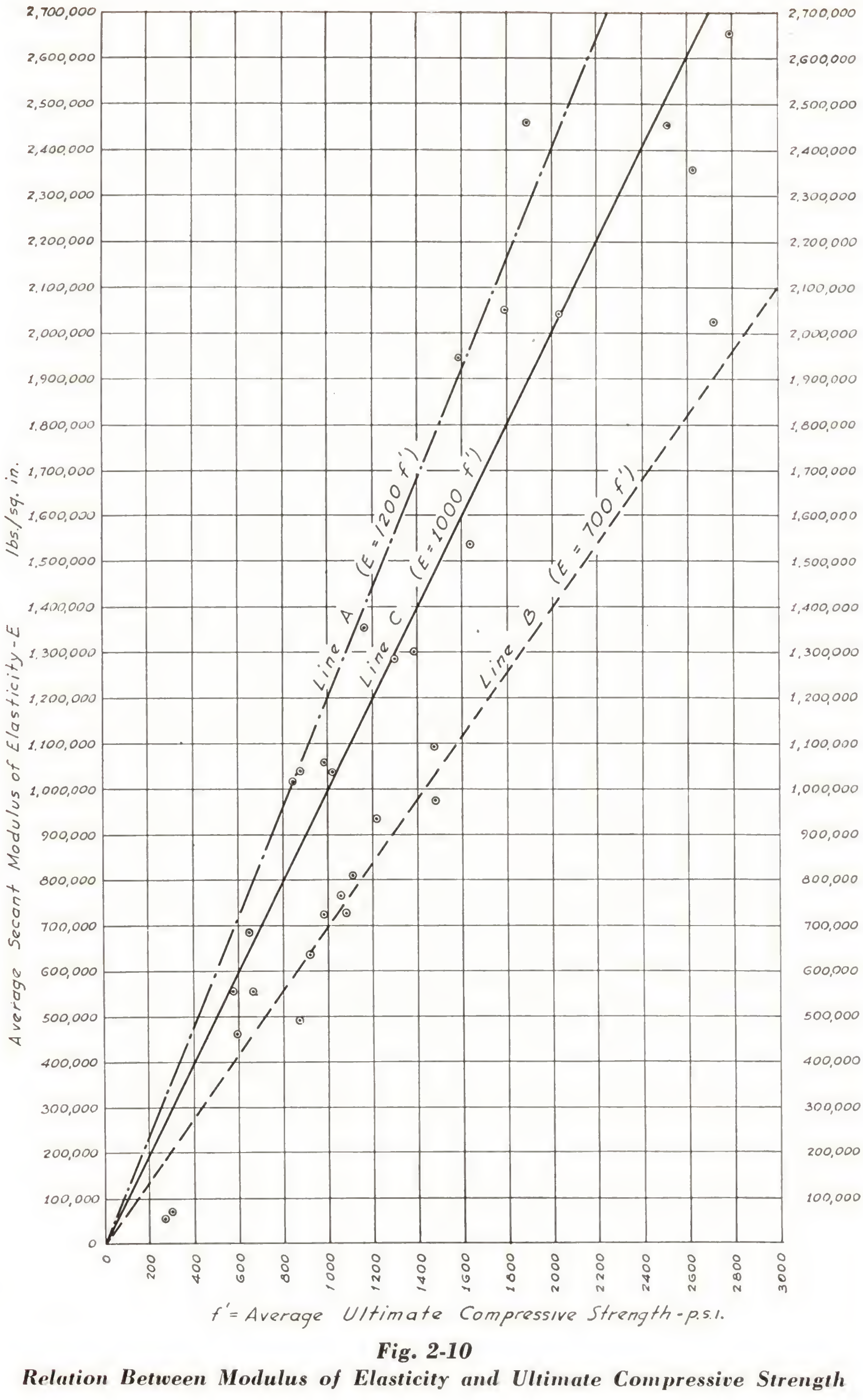


Fig. 2-10

Relation Between Modulus of Elasticity and Ultimate Compressive Strength

In the absence of test data on which to base a value of modulus of elasticity, it may be approximated by the equation $E = 1000 f'$ which is the equation for line C of Fig. 2-10.

As with compressive strength, modulus of elasticity in compression is affected by wall design, brick strength, mortar strength and workmanship. However, as indicated above, modulus of elasticity varies for different stresses, due to the plastic flow of the mortar, and is affected to a much greater degree than masonry strength by the number of mortar joints per unit of length in the test specimen.

In general, higher values of modulus of elasticity are associated with high strength brick, high strength mortars and Type A workmanship, characterized by complete filling of all mortar joints.

All other factors being equal, modulus of elasticity decreases with increase of total thickness of mortar joints per unit of length in a plane at right angles to the direction of the compressive force.

Calculated values of modulus of elasticity, derived from tests of reinforced brick flexural members and obtained from the straight line formulae used for the design of reinforced concrete and reinforced brick masonry, differ materially from the values of Fig. 2-10 obtained from compressive tests of brick masonry walls. This is due partly to the fact that straight line formulae for design are based on the assumption that the masonry carries no tensile stress which is incorrect until cracks develop in the masonry, and also to the difference in the distribution of stress between flexural and concentrically loaded compression members.

As in the case of ultimate compressive strength, recommended values of modulus of elasticity for use in design of reinforced brick masonry are based upon the modulus of elasticity of masonry piers built of similar materials (brick and mortar), the same type of workmanship and the same bonding arrangement as the proposed construction.

Table 2-30, compiled from data included in the report of Raymond E. Davis to the Clay Products Institute of California, discussed in Section 203 (b), gives values of modulus of elasticity calculated from the data obtained from tests of small brick beams. Regarding these tests, the report states: "Specimens, 8 by 4 by 32 in., were built as piers by laying brick flatwise one upon the other with ½-in. joints between. Specimens were wet cured for the first 14 days and then dry until tested at the age of 6 months.

"The specimens were tested as simple beams 4 in. deep on a 30-in. span with center loading. The center deflection was observed using two dial gages, one on either side of the beam.

"Load-deformation diagrams, which are not shown here, were plotted for each beam and the modulus of elasticity was computed for a flexural stress of 60 psi, this stress being slightly less than half the average modulus of rupture for all groups."

TABLE 2-30
MODULUS OF ELASTICITY IN FLEXURE
UNREINFORCED BRICK BEAMS

Mortar Mix①	No. Specimens	Modulus of Rupture, psi	Modulus of Elasticity psi
1:⅓:4	6	137	509,000
1:½:4½	5	112	477,000
1:1:6	3	97	559,000

① For properties of mortar, see Tables 2-7 and 2-8, Section 203(b).

206. REINFORCED BRICK MASONRY

As indicated in Chapter 1, research on reinforced brick masonry was initiated in the United States during the late 1920s largely as a result of the activities and sponsorship of the National Brick Manufacturers Research Foundation and the Common Brick Manufacturers Association. One of the earlier projects sponsored by these organizations was the erection of "demonstration structures" of reinforced brick masonry throughout the country. Most of these structures were built at brick plants and the object of constructing them, as stated in Report No. 1 of the Committee on Reinforced Brick Masonry of the National Brick Manufacturers Research Foundation, was "to stimulate interest among brick manufacturers, architects, engineers, mason contractors, building inspectors, etc., in improving brick masonry design by the construction and loading of two beams and one slab of reinforced brick masonry. Data obtained from this demonstration will be useful in making comparisons for masonry from various parts of the country, but it is not thought to obtain quantitative data such as would result in a well-equipped laboratory under the direction of scientifically trained men.

"This demonstration will have served its purpose if it stimulates the interest of those directly connected with the industry, as well as those active in building construction, in the great possibilities of this development and also their direct responsibility in furthering the scientific research necessary to prepare the data required by designers, building officials, etc., before the widespread use of this construction may be expected."

Figs. 2-11 and 2-12 are a plan and section of the demonstration structure and Fig. 2-13 shows one of these structures loaded to failure.

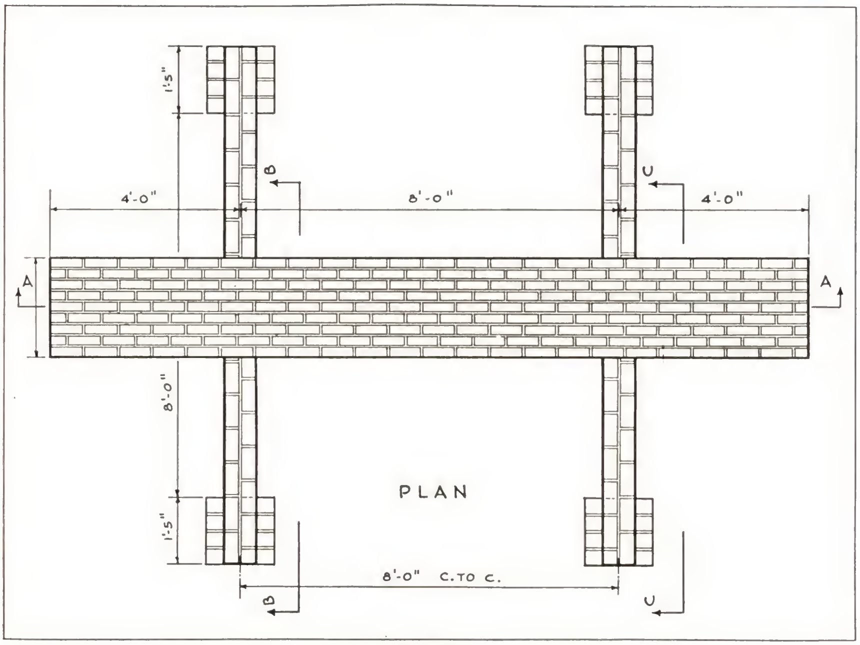
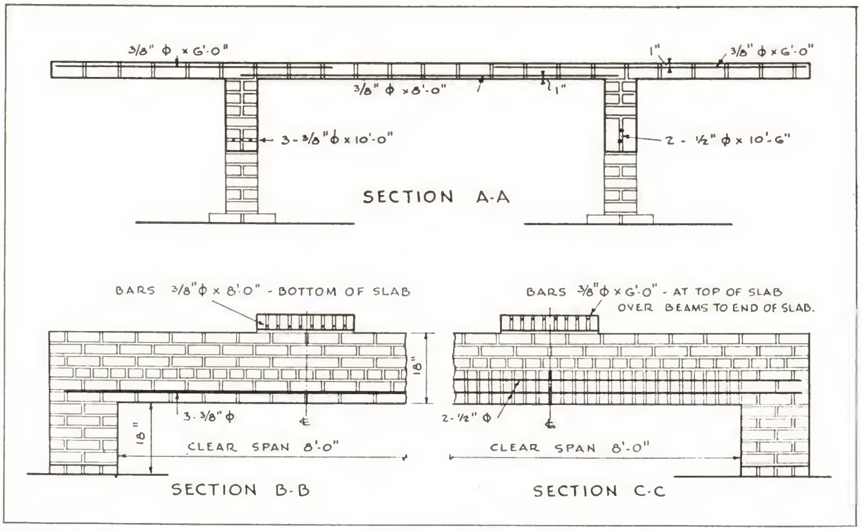


Fig. 2-11
Plan of Demonstration Structure



Note: Brick headers such as used in these beams should not be used in recommended grouted construction.

Fig. 2-12
Section of Demonstration Structure

Fig. 2-13
Demonstration Structure Loaded to Failure

These tests provided dramatic illustrations of the strength of reinforced brick masonry beams and slabs and undoubtedly contributed in creating the interest which has since developed throughout the design professions in reinforced brick masonry.

The construction and tests of these structures were not intended to simulate laboratory conditions and, consequently, many variables which have since been found to have an important effect upon the strength of reinforced brick masonry were not controlled. However, the data obtained from these tests are of value as an indication of the stresses which can be developed in reinforced brick masonry built under field conditions.

Tables 2-31 and 2-32, which are reproduced from Report No. 8 of the Committee on Reinforced Brick Masonry of the National Brick Manufacturers Research Foundation, summarize the data obtained from the tests of 20 "demonstration structures".

Fourteen different types of brick were used for the construction of these structures, the physical properties of which varied widely. Mortar, consisting of 1 part cement: 1 part hydrated lime: 6 parts sand by volume, was used in all of the structures except tests No. 1 and No. 7. The mortar used in these structures was 1 part cement: .15 part hydrated lime: 3 parts sand by volume, and 1 part cement: 2 parts sand by volume, respectively.

Regarding the data presented in Table 2-31, the report states: "Computed values of the shearing and compressive stresses in the masonry, the stress in the steel, and the bond stress are based on an assumed value of the depth to the neutral axis of 0.38 of the depth to the steel and 0.88 of the depth for the values of the distance from center of steel to centroid of compression.

"The load producing the maximum allowable deflection for plastered ceilings of 1/30 in. per ft. of span is of particular interest since it is the limiting feature of design. This deflection was not obtained for any slab for a live load of less than 300 lb. per sq. ft. which is required to stress the steel to the usual design value of 20,000 psi. Assuming this working stress, the maximum possible design bending moment is 48,500 in.-lb. With loads producing this maximum design bending moment, 13 slabs showed corresponding deflections less than the allowable. The remaining 7 were apparently deficient in some manner; 3 of these are known to have had poor sand in the mortar Those slabs that did not fail at small loads gave evidence of satisfactory performance.

"The greatest load was carried by Test XV with 20,530 lb. or 1412 lb. per sq. ft. Loads ranged from this value down to 5640 lb. or 350 lb. per sq. ft. Omitting those structures with known weaknesses, the remaining ones indicate strengths far beyond design requirements. Apparently an increase in age beyond the usual 28 days produces a corresponding increase in strength. Tests XI and XII at 34 days showed less than Tests XIV and XV using the same materials at 54 days.

"The calculated values are of interest in a comparative respect. In no case did any of the slabs fail in compression. Figured stresses range from 1030 psi to 4090 psi. These values are far beyond the usual accepted allowable working stresses for any kind of brick masonry. Steel stresses in many cases approach the yield point, with a maximum in Test XV of 63,000 psi. The fact that most slabs developed 20,000 psi or more in the steel indicates that the actual shearing, compressive, and bond stresses were not excessive. Unit

TABLE 2-31
SUMMARY OF TESTS OF SLABS

Test No.	Center Slab Max. Load lb.	Center Slab Max. Load psf.	Center Slab Max. Defl. in.	End Slabs Max. Load lb.	End Slabs Max. Load psf.	Age days
I	14620	995	0.219	7300	995	102
II	12120	825	1.203	1364	186	38
III	12300	837	.766	1395	191	48
IV	6780	461	.125	1410	96	46
V	6598	434	.615	1739	227	37
VI	6924	452	.500	1739	227	36
VII	16440	1082	—	4110	536	37
VIII	10300	700	.375	2700	371	48
IX	10230	698	.734	1980	270	41
X	11400	730	.438	2850	366	118
XI	16206	1073	.650	4435	554	34
XII	9677	640	.260	2688	336	34
XIII	7245	442	.280	1807	219	50
XIV	19245	1324	.50	4815	660	56
XV	20530	1412	.46	5136	704	54
XVI	8900	556	.086	4450	556	184
XVII ①	3630	220	.03	3630	220	184
XVIII	7260	468	.375	1812	234	114
XIX	7860	495	.19	1935	240	31
XX	5640	350	.19	1410	170	30

① Damaged during test, not considered in discussion.

SUMMARY OF TESTS OF SLABS (Continued)

Test No.	Center Slabs Max. B. M. in.-lb.	Max. Shear psi	Max. F_b psi	Max. F_s psi	Max. Bond Stress psi	Unit Load First Crack	Unit Load Defl.=0.26 in.
I	85000	110	2800	40000	320	350	Failed
II	122096	92	4030	57000	267	413	332
III	124000	93	4090	58300	270	608	456
IV	47500	51	1500	22300	149	—	Failed
V	39590	50	1250	18500	145	332	340
VI	41500	53	1310	19500	153	332	333
VII	98640	124	3060	46300	361	457	645
VIII	64700	78	2135	30400	226	448	612
IX	61380	77	2020	28800	251	405	540
X	68400	86	2000	32100	229	423	596
XI	95500	125	2800	45000	358	560	720
XII	55800	75	1640	26400	214	560	640
XIII	47500	56	1400	22400	160	306	408
XIV	12500	153	3800	58500	427	792	968
XV	134000	163	4080	63000	455	880	880
XVI	107000	71	3250	50300	197	—	—
XVII①	87000	29	2650	41000	80	—	—
XVIII	43560	58	1325	20400	160	468	390
XIX	47160	—	—	—	—	495	495
XX	33840	45	1030	15800	125	350	350

① Damaged during test, not considered in discussion.

TABLE 2-32

SUMMARY OF TESTS OF BEAMS

Test No.	Max. Load lb.	Max. Bending Moment in.-lb.	Max. Shear lb.	Soldier Beams				
				Max. Defl. in.	Max. F_s psi	Max. F_b psi	Max. v psi	Max. u psi
I	14610	306810	7305	0.095	64000	1170	65	190
II	7424	155900	3712	.250	32400	592	33	96
III	7545	158445	3773	.063	33000	605	33	98
IV	4800	100800	2400	.063	22500	384	21	62
V	5038	105738	2519	.003	22000	403	22	65
VI	5201	109401	2600	.000	22800	418	23	67
VII	12330	258930	6165	.000	54000	985	55	160
VIII	7850	164850	3925	.086	34300	628	35	102
IX	5940	124740	2970	.032	26000	475	26	77
X	8400	176400	4200	.063	36700	673	37	109
XI	4704	96500	2352	.15	19500	369	21	61
	10351	217000	5176	—	—	—	—	—
XII	7527	158000	3764	.163	31800	603	34	96
XIII	5432	114000	2716	.078	23000	435	25	70
XIV	14437	303200	7219	.175	61500	1151	66	187
XV	15400	323400	7700	.29	—	—	—	—
XVI	8900	186900	4450	.109	37900	710	40	115
XVII	3630	76230	1815	.01	15450	290	16	47
XVIII	6442	114280	2721	.046	23200	435	25	70
XIX	5865	123190	2933	.078	25000	468	37	76
XX	4230	88830	2115	.008	18000	338	19	55

SUMMARY OF TESTS OF BEAMS (Continued)

Test No.	Max. Load lb.	Max. Bending Moment in.-lb.	Max. Shear lb.	Running Bond Beams				
				Max. Defl. in.	Max. F_s psi	Max. F_b psi	Max. v psi	Max. u psi
I	14610	306810	7305	0.063	68300	953	59	152
II	7424	155900	3712	.219	34600	482	30	77
III	7545	158445	3773	.032	35200	492	30	78
IV	4800	100800	2400	.203	24000	313	19	50
V	5038	105738	2519	.156	23400	329	20	53
VI	5201	109401	2600	.000	24400	341	21	54
VII	12330	258930	6165	.000	57500	804	50	129
VIII	7850	164850	3925	.071	36500	512	32	82
IX	5940	124740	2970	.047	27800	388	24	62
X	8400	176400	4200	.032	39200	550	34	88
XI	4704	96500	2352	—	—	—	—	—
	10351	217000	5176	—	48800	675	42	108
XII	7527	158000	3764	.093	35400	492	31	78
XIII	5432	114000	2716	.063	25400	335	23	57
XIV	14437	303200	7219	.18	67500	945	58	150
XV	15400	323400	7700	.28	71800	1008	62	160
XVI	8900	186900	4450	.04	41500	583	36	92
XVII	3630	76230	1815	.03	16900	237	15	38
XVIII	6442	114280	2721	.31	25400	356	22	56
XIX	5865	123190	2933	.016	27350	384	24	61
XX	4230	88830	2115	.016	19700	277	17	44

shearing stresses ranged from 45 psi to a maximum of 163 psi in Test XV. Unit bond stresses ranged from 125 psi to 455 psi. The unit load at first crack ranged from 306 to 880 lb. per sq. ft. Permitting the load to remain overnight in Tests XIV, XV, XVI and XVII showed that in those cases there was no apparent plastic flow."

Data presented in Table 2-32, in general, do not represent ultimate values. The report states: "All of the beams were not tested to failure. Seven beams were actually loaded to failure, 2 in Test IV, 2 in Test XI, one in Test XIV, one in Test XV, and one in Test XIX. The beams in Test XII showed signs of cracking but did not actually fail. The remaining beams carried varying loads ranging from 3630 to 14,610 lb.

"In considering the results of these beam tests it is important to keep in mind the original purpose. The beams were designed to represent lintels usually loaded with a uniform dead wall load and in some cases a concentrated beam load. The amount of reinforcing is small. These beam tests demonstrate without a doubt that ordinary masons using recognized mortar mixes and brick from the existing market can produce a new form which is capable of performing in ways hitherto impossible for brick masonry. Even those beams that failed carried sufficient load without excessive deflection to warrant their use in ordinary lintel construction. Values of computed stresses are given only for comparison. No deformations were measured."

(a) Basis for Working Stresses. Tests of reinforced brick masonry flexural members have indicated that the compressive strength of brick masonry prisms, having a height to thickness ratio of not less than 2, constructed of similar materials, with the same type of workmanship and with the same bonding arrangement as the proposed construction, are reasonably accurate measures of the ultimate compressive strength of the masonry in flexure. Data indicating this relationship will be presented in Subsections (b) and (e).

In constructing the test specimens, it is important that the arrangement of the brick with relation to the compressive force be the same as that which will exist in the flexural member. In such members, compressive force may be applied normally to any one of the three faces of the brick: (1) the ends, as when brick are laid in running bond in a lintel or beam; (2) the beds, as in typical wall construction when the wall is subjected to lateral force; (3) the edges (faces), as headers in a beam or lintel or as a rowlock course in wall construction.

TABLE 2-33

COMPRESSIVE STRENGTH OF BRICK

Type	Compressive Strength, psi		
	Flatwise	Edgewise	Endwise
End cut, clay	2586	2390	5550
Side cut, shale, 3 cores	12500	5860	5900
Side cut, shale, 16 cores	9195	4034	3970
End cut, clay	4510	5240	4200

Table 2-33 shows the compressive strength of 4 types of brick when tested on end, on edge and flat. It will be noted from this table that there is no consistent relationship between these values for different brick. This variation is due to different methods of manufacture and to different raw materials.

The value of modulus of elasticity of masonry used in the design of reinforced brick masonry flexural members is assumed at 1000 times the ultimate strength of brick masonry prisms constructed as described above.

While, as indicated in Section 205(d), there does not appear to be a consistent relationship between the ultimate strength of masonry and modulus of elasticity, the value assumed is a reasonable approximation and checks fairly closely with values computed from test data using straight line formulae.

As indicated in Bureau of Standards Research Paper 504, "Shear Tests of Reinforced Brick Masonry Beams", by D. E. Parsons, A. H. Stang and J. W. McBurney: "From the standpoint of design, accurate values of n are rarely required for satisfactory results as an error of 50 per cent in the assumed value of n would rarely cause an error of more than 15 per cent in the calculated stress in the masonry or more than 4 per cent in the calculated stress in the steel."

(b) Reinforced Brick Beams. In Technical Paper No. 38, "Notes on Reinforced Brickwork", by A. Brebner, which was published by the Public Works Department of the Government of India in 1923, the author reviews data obtained from tests of numerous reinforced brick beams and slabs and concludes: "Reinforced brick beams and slabs may be designed according to reinforced concrete theory."

Following publication of this report, numerous tests were conducted in the United States of reinforced brick slabs and beams to test the validity of this conclusion when applied to the materials and methods of construction commonly used for unreinforced masonry.

These tests also provided data on the ultimate strength of reinforced brick masonry members and, more importantly, indicated changes in the design and methods of construction of flexural members from those originally proposed which might be expected to increase the strength and efficiency of the construction. As a result of the latter, grouted reinforced brick masonry eliminating headers was developed. Many factors affecting the strength of bond between mortar and brick were identified and recommended procedures for controlling these factors in construction were developed.

Since the materials (particularly the mortar) and the method of constructing the test specimens of the earlier tests differ from current recommendations, the ultimate strengths obtained cannot be considered as representative of present day construction which, in general, will develop considerably higher strength. However, these tests did verify the validity of the conclusion that the formulae used for the design of reinforced concrete flexural members are also applicable to the design of reinforced brick masonry, and that stresses computed by these formulae are in close agreement with observed values.

In the following brief review of some of these early tests, the data included in the reports will not be reproduced but only conclusions will be quoted. For the data, the reader is referred to the original reports, all of which have been published.

University of Wisconsin. The paper, "Tests of Brick Masonry Beams", by M. O. Withey, published in the Proceedings of the American Society for Testing Materials, Vol. 33, Part II, 1933, reports tests of 25 reinforced brick beams 8 in. by 12 in. in cross-section, tested under third point loading on an

8-ft. span. Longitudinal steel reinforcement varied from 0.5 to 2.3 per cent and several percentages of stirrup reinforcement were used. The mortar consisted of 1 part high-early-strength portland cement: .78 parts hydrated lime: 4.7 parts sand by volume.

Deformeter readings were taken from which the position of the neutral axis could be determined. Observed value of k (ratio of depth of neutral axis from top of beam to depth to center of reinforcement) were compared with calculated values based on the straight line formula for one-third the maximum load and on the parabolic formula for the ultimate load.

The modulus of elasticity of the masonry used in determining n was obtained from tests of brick piers, 8 in. by 8 in. by 25 in. high, constructed of similar materials, with the same type of workmanship and the same bonding arrangement as the compressive section of the beams.

With few exceptions, observed and calculated values of k at both one-third maximum load and at ultimate load were in close agreement.

Most of the beams failed by tension in the reinforcement. However, three specimens are reported as failing, due to compression in the masonry; and in many of the tensile failures, the calculated compressive stress in the masonry apparently approached the ultimate.

Assuming the maximum compressive strength of the masonry as the strength of the brick masonry piers described above, the report states: "By using adequate percentages of longitudinal steel and stirrups, the full compressive strengths of both the Waupaca and Streator brick masonry were developed."

In the conclusions, the author states: "Where web reinforcement is required and no longitudinal rods are bent up, a stirrup spacing approximately one-half the effective depth of the beam is satisfactory.

"The tests indicate that the formulae used in the calculation of fiber, shear, and bond stresses and the deflections for reinforced concrete beams can, with proper constants, be used in like calculations for reinforced brick beams.

"In order to secure good strengths, particular attention must be given to see that all joints are completely filled.

"In designing the arrangement of coursing, it is preferable to eliminate headers from heavily compressed portions of beams.

"A knowledge of the moisture content desirable in a given brick and of the bond of the mortar to it is essential to proper design of reinforced brick beams."

National Bureau of Standards. Tests to determine the resistance to shear of reinforced brick masonry beams are reported in Bureau of Standards Research Paper 504, "Shear Tests of Reinforced Brick Masonry Beams", by D. E. Parsons, A. H. Stang and J. W. McBurney, 1932. Eighteen beams, approximately 1 ft. sq. in cross-section, were tested on a 12-ft. span by 2-point loading at the quarter points. Each beam contained six ½-in. sq. deformed bars as tensile reinforcement and no web reinforcement. The mortar consisted of 1 part portland cement: .15 parts hydrated lime: 3 parts sand by volume.

Maximum shearing stresses, based on experimentally determined values of k, ranged from 65 to 159 psi. The report states: "Tensile and shear strengths of the masonry were closely related to the resistance to diagonal tension of the masonry beams."

The tensile strengths referred to are discussed in Section 203(c).

Virginia Polytechnic Institute. Thirty reinforced brick masonry slabs, 3¾ in. deep (1 brick on edge), were tested at Virginia Polytechnic Institute and the results reported in Engineering Experiment Station Bulletin No. 9, 1932, "An Investigation of the Performance Characteristics of Reinforced Brick Masonry Slabs", by John W. Whittemore and Paul S. Dear.

Mortar used in the construction of these slabs consisted of 1 part cement, 1 part hydrated lime and 6 parts sand by volume. Reinforcement consisted of four ¼-in. plain round bars in a slab width of 14 in. Two-point loading was applied at the third points of the span. Deflections were observed for increments of loading, and stresses in the masonry and steel were calculated.

The authors conclude: "In practically every case during the testing of the 30 slabs, the initial failure occurred before the allowable deflection was reached. The instantaneous recovery performance of the slabs, even well past the design load, was seen to be very favorable. The deflection performance of the slabs was also very good. These facts warrant the conclusion that all slabs tested during the investigation possessed ample stiffness until well past the design load.

"Under ordinary circumstances of slab design, the compressive strength and the transverse strength of the individual brick are apparently of minor importance in the performance of the slab. As a general rule, any well fired brick has sufficient strength to introduce an ample factor of safety in respect to the ultimate strength of the brick masonry.

"In reinforced brick masonry construction, the percentage of absorption of the individual brick assumes the greatest importance of the usually determined physical properties of the brick.

"The strength of adhesion of mortar to brick and of mortar to steel are very important items.

"During the investigation it was noted that the best slab performances were associated with brick having the highest percentage of absorption and having the greatest strength of adhesion of mortar to brick. This indicates that there is a direct relationship between absorption and mortar joint strength." (Note: All brick were wetted "by immersion in water for approximately 15 min." just prior to laying. Brick absorption by 48-hr. immersion in cold water ranged from 4.97 per cent to 11.98 per cent. Wetting of low absorption brick was unnecessary and probably reduced bond strength between mortar and brick.)

"Surface characteristics and texture of the individual brick are important factors in the performance of reinforced brick masonry, since they influence mortar joint strength. Brick with roughened surfaces exposed to union with the mortar greatly aid mortar joint strength and slab performance.

"Reinforced brick masonry slabs constructed by grouting the mortar joints are apparently well able to withstand design loads. The widely used 1:1:6 mortar mix is adaptable to the construction of slabs by grouting the mortar joints.

"Though plain round reinforcing rods are adaptable to reinforced brick masonry slab construction, it is the opinion of the investigators that deformed rods would be more desirable since they would promote greater bond strength between the mortar and the steel.

"All brick should be wetted before use in reinforced brick masonry. The degree of wetting should be governed by the absorption of the individual

brick. Brick with high absorption should be wetted more thoroughly than brick with low absorption in order to prevent the destruction of the mortar strength by decreasing the water-cement ratio.

"Quality of workmanship is an important factor in slab performance.

"The actual stresses in steel and brickwork, as experimentally determined, are well below those calculated by reinforced concrete design formulae.

"The experimentally determined values of k, j and n closely agree with the theoretical values.

"It is evident that the formulae of reinforced concrete are adaptable, with slight modifications, to reinforced brick masonry design.

"Reinforced brick masonry slabs perform in a very similar manner to reinforced concrete slabs and are, therefore, theoretically and experimentally practicable."

Subsequent to these tests, the same investigators tested 4 reinforced concrete slabs, the results of which are reported in Virginia Polytechnic Institute Engineering Experiment Station Bulletin No. 15, 1933. The purpose of these tests is stated in the report as follows:

"Previous investigations by the authors led to several conclusions regarding reinforced brick masonry slabs. One of these conclusions, 'Reinforced brick masonry slabs perform in a very similar manner to reinforced concrete slabs, and are, therefore, theoretically and experimentally practicable,' was the basis of this investigation.

"In order to substantiate further this conclusion, tests were made on four reinforced concrete slabs having the same size, type of reinforcement, percentage of steel, and effective depth as possessed by the slabs of reinforced brick masonry previously tested. The methods of curing and the methods of testing the concrete slabs were also identical. A true comparison of the various performance characteristics was therefore possible."

Data similar to those obtained from tests of reinforced brick masonry slabs were obtained in this investigation and the authors conclude:

"Over ranges of loading well past design loads, the slabs of both types of construction perform like homogeneous beams in that all relations of load and moment to deflection and stress are linear.

"The slabs of both types of construction possess ample stiffness even well above design loads.

"From the viewpoint of induced stresses, both types of construction exhibit very amply safety factors when reinforced concrete theory is used as the basis of design.

"The various performance characteristics of both types of construction are strikingly similar and in very close agreement throughout the entire range of loading.

"The formulae of reinforced concrete design are adaptable and applicable to the design of reinforced brick masonry slabs.

"The conclusion which formed the basis of this investigation—'Reinforced brick masonry slabs perform in a very similar manner to reinforced concrete slabs and are, therefore, theoretically and experimentally practicable'—is confirmed fully and completely."

University of Southern California. Probably the most recent tests of reinforced brick masonry beams from which published data are available were conducted at the University of Southern California during 1951 under the sponsorship of the Associated Brick Manufacturers Association of South-

ern California. The report of these tests, by Robert R. Schneider, was published in the December 28, 1951, issue of Southwest Builder and Contractor.

A major part of this investigation was devoted to the determination of shearing strength, as a measure of resistance to diagonal tension, of reinforced brick masonry beams and to the evaluation of the factors affecting this property.

Test specimens were constructed in accordance with currently recommended construction practices (see Chapters 3, 7 and 8) and the results provide valuable data on which to base working stresses. Shear is frequently a critical factor in the design of reinforced brick masonry beams and, for this reason, the information included in the Schneider report is reviewed in some detail.

Twenty-seven beams designed to fail in diagonal tension were tested. Eighteen of these were constructed of "common" brick and 9 of "face" brick. The physical properties of the brick are shown in Table 2-34.

TABLE 2-34

PHYSICAL PROPERTIES OF BRICK

Type	Average Modulus of Rupture, psi	Average Compressive Strength, psi	Average Absorption, %	
			24-hr. Cold	5-hr. Boil
Common	660	2364	20.18	24.6
Face	776	4216	12.2	14.5

Two mortar mixes were used; one designated as Type A consisting of 1 part cement: .16 parts hydrated lime: 3 parts sand by volume, and the other designated as Type B consisting of 1 part cement: 3/10 parts hydrated lime: 4½ parts sand by volume. The lime was a highly plastic, pressure hydrated hydrate.

Regarding the reinforcement, the report states: "Two types of reinforcing steel were provided. The ASTM Specifications A305- deformed bar, ½-in. diameter, was used for the bond tests (discussed in Section 203(d)) and also for the web reinforcement. All other reinforcing consisted of the standard deformed bar."

Two types of workmanship were employed in constructing the specimens. The report states: "In order that the effects of workmanship could be evaluated, beams involving both 'good work' and 'poor work' were constructed. Good work indicates proper workmanship methods, such as those required by the Los Angeles City Building Code where continuous inspection is utilized. Poor work simulated that type of workmanship which might be obtained in unsupervised construction; open head joints, deep furrowing, bricks not 'shoved' and grout not puddled.

"The results obtained in the tests must be evaluated in the light of these field conditions. These were not controlled laboratory tests. Representatives from the supervising institution were present throughout the construction process and it was noted that actual job conditions prevailed during the entire construction period."

TABLE 2-35

DESCRIPTION OF TEST BEAMS

Designation	Construction
A	Common brick, A mortar, poor workmanship
B	Common brick, A mortar, good workmanship
AA	Common brick, B mortar, poor workmanship
BB	Common brick, B mortar, good workmanship
AF	Face brick backed with common brick, A mortar, good workmanship, ⅝-in. head joints and ½-in. bed joints
BF	Face brick backed with common brick, A mortar, good workmanship, ¼-in. head joints and ¼-in. bed joints
CF	Face brick backed with common brick, A mortar, good workmanship, head joints brick to brick, no mortar, and ½-in. bed joints
H	Common brick, A mortar, good workmanship
J	Common brick, A mortar, good workmanship
Reinforcement	Beam H reinforced with seven ⅝-in. deformed round bars, no web reinforcement
	Beam J reinforced with fourteen ⅝-in. deformed round bars and fourteen ½-in. deformed round stirrups spaced 3 in. center to center from each end
	All other beams reinforced with six ⅝-in. deformed round bars, no web reinforcement

Fig. 2-14 shows cross-sections and elevations of the various types of beams and the significant variables in the construction are listed in Table 2-35. All beams were tested on a span of 55½ in. under two-point loading as indicated in Fig. 2-14.

Calculated stresses at ultimate load, as listed in the report, are given in Table 2-36. Values reported are the average of 3 tests.

All specimens failed by diagonal tension and, quoting from the report, "All of the theoretical computations were made with the usual reinforced concrete design and investigation equations. These computations were predicated on the assumption that the grout on the tension side of the flexural member had cracked up to the botttom layer of longitudinal steel."

Regarding the design of the test specimens, the report states: "Since the primary purpose of the flexural tests was to measure diagonal tensile resistance in the reinforced grouted brick masonry beams, the longitudinal steel was purposely over-designed so that this property could be investigated."

As a result of this design, tensile stresses in the steel, compressive stresses in the masonry and bond stress between masonry and reinforcement developed at ultimate load are relatively low as compared to stresses developed in beams having adequate web reinforcement.

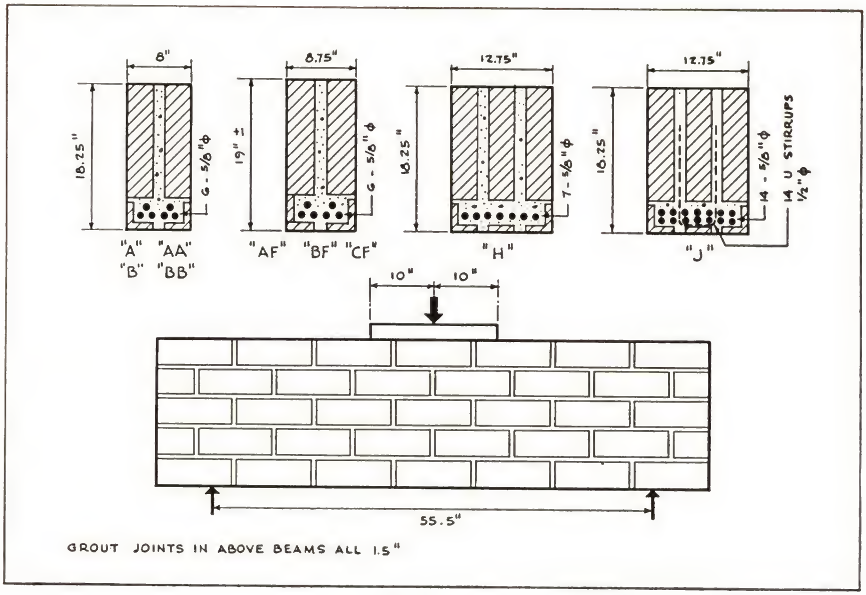


Fig. 2-14
Details of Test Specimens

TABLE 2-36

STRENGTH OF RGBM BEAMS

Specimen No.	Stress, psi			
	f_s	f_m	v	u
Two-Wythe				
A	10043	547	118	80
B	15066	819	177	120
AA	15283	833	177	121
BB	19233	1049	225	153
AF	16300	887	176	125
BF	15933	803	173	127
CF	17500	845	188	139
Three-Wythe				
H	26033	1160	224	209
J	24433	1717	408	194

f_s is unit tensile stress in steel
f_m is unit compressive stress in masonry
v is unit shearing stress in masonry
u is unit bond stress between steel and masonry
Web reinforcing in the J series consisted of fourteen ½-in. round stirrups at 3 in. c-c.

The shear stresses, however, may be considered as a measure of the resistance which the masonry offers to diagonal tension.

Effect of Mortar. Beams A and B and AA and BB were similar in all respects except the mortar used in their construction. It will be noted from Table 2-36 that the average shearing stresses of the beams constructed with B mortar (1 cement: 3/10 hydrated lime: 4½ sand by volume) were greater when both good and poor workmanship were utilized than the stresses developed by the beams in which A mortar (1 cement: .16 hydrated lime: 3 sand by volume) was used.

The average ultimate shearing stresses for the A mortar specimens were approximately 79 per cent of the values obtained with the B mortar specimens where good workmanship was utilized and 67 per cent where poor workmanship was used.

These data are further evidence of the conclusion indicated by the data on bond strength of mortar to brick discussed in Section 203(c), and the data on transverse strength of masonry discussed in Section 205(b) which indicate an optimum ratio of lime to cement of between ¼ and 1 in mortars which develop maximum bond strength with brick.

Effect of Workmanship. A comparison of the ultimate shearing stress of specimens A with specimens B and of specimens AA with specimens BB gives an indication of the effect of type of workmanship on shearing strength. It will be noted that poor workmanship specimens developed ultimate shearing stresses approximately 67 per cent of those obtained with good workmanship when A mortar was used, and approximately 79 per cent of the good workmanship values when B mortar was used.

The report states: "Particular care was taken to insure that the poor work specimens were actually such; that is, deep frog joints, bricks not shoved, etc."

Effect of Head Joint. Beams AF, BF and CF were designed to determine the effect of width of head joint on shearing strength, also the effect of using brick having different physical properties. It will be noted that the average shearing stress of the 3 types tested are 176 psi, 173 psi and 188 psi for specimens with ⅝-in., ¼-in., and no (brick to brick, no mortar) head joints.

Effect of Brick. Comparing the shearing stresses developed by face and common brick beams with the common brick beams constructed with A mortar and good workmanship, the average shearing stress developed by the common brick beams was 177 psi and by the combination face brick and common brick beams 179 psi (average of 3 groups).

The report states: "There was no apparent shifting of the load to either the common or face brick and the two types of brick behaved as a unit throughout the loading period."

Three-Wythe Beams. Table 2-36 gives the computed stresses developed in the 3-wythe beams with and without web reinforcement. It will be noted that the ultimate shearing stress developed in the 3-wythe beam without web reinforcement is approximately 25 per cent greater than the stress developed in similar 2-wythe beams. The average shearing stress of 408 psi developed by the beams with web reinforcement is 2.3 times the average shearing stress developed by 2-wythe beams without web reinforcement and 1.8 times the stress developed in the 3-wythe beam without web reinforcement.

The following is quoted from the summary and conclusions of the report: "The 1:4½:0.30 B mortar (1 part by volume cement to 4½ parts sand and to

0.30 parts lime) yielded a higher average shearing stress, 225 psi, than the 1:3:0.16 A mortar mix, for which the average stress was 177 psi. Both of these values were obtained with the 'proper workmanship' specimens.

"With 'improper workmanship' methods, the 1:4½:0.30 mortar yielded the higher shear values. The average unit shear stress for the 'poor work' beams was 118 psi for the 1:3:0.16 mortar, and 177 psi for the 1:4½:0.30 mortar specimens.

"The compressive stress in the masonry was not critical, since only a part of the ultimate compressive strength of the individual brick was developed.

"The combined face and common brick specimens behaved as a unit throughout the loading period, and there was no apparent shift of the load to either the common or the face brick. The average unit shearing stress obtained was 179 psi, which compares with the 177-psi average obtained on the two-wythe common brick units, the 1:3:0.16 mortar and 'proper workmanship' methods being used in both types.

"No apparent difference in flexural strength due to head joint size was noted in the combined face and common brick specimens.

"During the initial loading stage (up to 10,000 lb.), the mid-span deflections of the two-wythe beams increased uniformly at the rate of approximately 0.001 in. per 1000 lb.

"The 1:4½:0.30 mortar specimens offered slightly greater resistance to deflection than did the 1:3:0.16 mortar beams. Likewise, the 'good work' specimens were stiffer than the 'poor work' specimens. The least resistance to deflection was obtained with the 'poor work', A mortar beams.

"All mid-span deflections at the ultimate load were less than the maximum allowable 1/360th of the span.

"The average ultimate shearing resistance of the three-wythe beams was 224 psi without web reinforcement, and 408 psi when the ½-in. stirrups, spaced 3 in. center to center, were used."

(c) Spacing of Wall Reinforcement. A reinforced brick masonry wall was tested in 1936 to determine the effect of the spacing of vertical reinforcement. This test was conducted under the supervision of the Smith-Emery Company of Los Angeles and the results were reported to the Clay Products Institute of Southern California by Charles H. Fork, Structural Engineer.

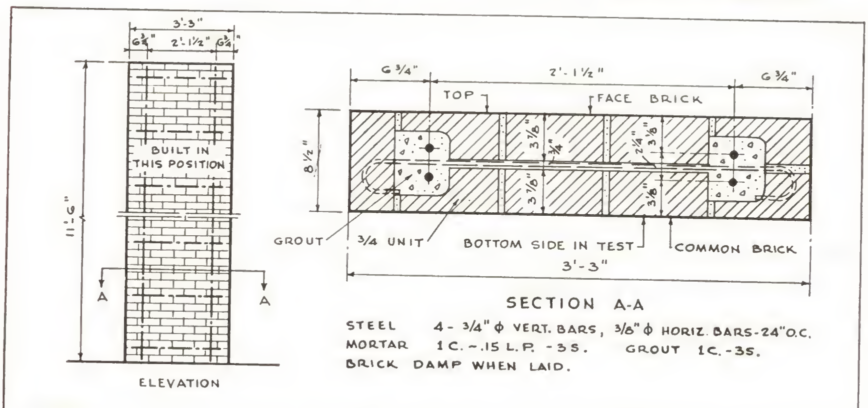


Fig. 2-15
Details of Test Specimen

Fig. 2-16
Test Specimen Under Load

The purpose of the test, as stated in Fork's report, was "to demonstrate the structural effectiveness when definite channels are provided for the reinforcing steel. In order to promote efficiency of bricklaying production, it was decided to keep a large spacing of reinforcing bars; 2 ft. 1 in. on centers in this case. The test will show whether the masonry is capable of spanning from bar center to bar center. The test will also demonstrate the effectiveness of a wall to take bending stresses when constructed of face brick in connection with common brick."

Fig. 2-15 is an elevation and cross-section of the test specimen showing the amount and location of the reinforcement, and Fig. 2-16 is a photograph of the specimen under maximum load.

Regarding the construction, the report states: "The wall was constructed at the yard of the Los Angeles Brick & Clay Company on April 24-25, 1936. The work was done by a mason hired by the day, under the author's supervision. The wall was built in an upright position in the usual manner of building walls. The ¾ unit was made by the bricklayer by clipping a regular brick with a trowel. Brick were laid up in the usual manner, using mortar for the bed joints and at the outer edges of the vertical joints. Two courses were laid up and then the interior space was poured full of thin flowing grout. The grout was puddled to insure settling. Brick were damp when laid. The details show the construction. The only headers used were at the ends of the wall. The wall was built up to a height of 11 ft. 3 in. The length is 3 ft. 3 in. and the thickness is 8½ in.

"The Los Angeles Brick & Clay Products Company's face brick were used on the compression side of the test piece. The vertical faces of the brick adjacent to the grouted joint were smooth. Brick were medium burned. The common brick were a very good grade and manufactured by the Higgins Brick Company.

"All mortar and grout were proportioned by volume measurement. The mortar mix is 1 part cement, .15 parts lime putty and 3 parts sand. The grout mix is the same except the lime putty is left out. Mortar was mixed by hand in a mortar box. Grout was mixed by hand in a bucket to a fluid consistency. Cement, lime putty and sand were of the usual commercial brand.

"The wall cured in open weather. Once a day for the first 20 days the wall was sprinkled."

In testing the specimen, increments of load were applied and the deflection observed. The load was then removed and the recovery of the slab determined. It was found that the performance of the wall was typical of reinforced concrete or reinforced brick flexural members; that is, a permanent set was found after the first application of the load, but upon re-application of the load, if it did not exceed the first loading, the deflection of the wall was only a little more than it was at the initial application of the load.

The maximum load applied to the wall including the weight of the test specimen was 738 lb. per sq. ft. Calculated stresses based upon two assumed values of n are given in Table 2-37.

TABLE 2-37

CALCULATED STRESSES

Load per lin. ft. lb.	Stresses, psi								Remarks
	Steel f_s		Brickwork f_b		Bond u		Shear v		
	n=		n=		n=		n=		
	20	37.5	20	37.5	20	37.5	20	37.5	
290	9350	9700	233	190	61	63	7.4	7.4	Test piece
538	17030	17700	425	348	111	115	13.5	14.0	Test piece & 4 courses
1878	60200	62500	1510	1240	395	410	48.0	48.5	Test piece & 30 courses
2463	79700	82300	1980	1620	520	538	63.0	65.0	Test piece & 41 courses

(d) Plastic Flow. Few data are available on the plastic flow of reinforced brick masonry structures. However, in 1949 tests were conducted by Anton Disch at the University of Wisconsin under the supervision of Professors K. F. Wendt, and G. W. Washa to determine the plastic flow of three types of masonry beams having high span to depth ratios. The report of these tests constitutes a thesis submitted in partial fulfillment for the requirements of the degree of Master of Science by Anton G. Disch.

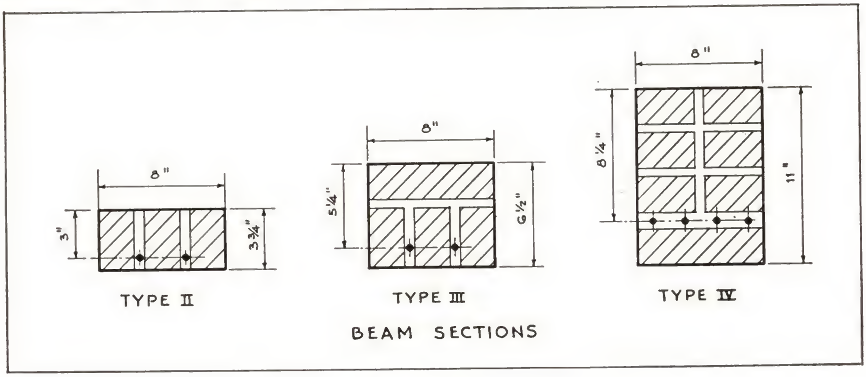


Fig. 2-17
Details of Test Beams

Fig. 2-17 shows a cross-section of the beams which were tested on spans ranging from 8 to 21½ ft. The variables introduced into the construction were two types of brick and two types of mortar. Two specimens of each type of construction were tested.

The program also included tests of brick masonry piers to determine ultimate strength and modulus of elasticity of the masonry, and plastic flow of the piers.

Table 2-38 gives the physical properties of the brick used in the construction and, as will be noted from this table, there was a wide variation

TABLE 2-38
PHYSICAL PROPERTIES OF BRICK

Brick Type	Absorption per cent		Suction gm/min/ 30 sq. in.	Modulus of Rupture psi Flatwise	Compressive Strength, psi		
	24-hr. Cold	5-hr. Boil			Flatwise	On Edge	On End
Streator Light	6.63	7.06	15.9	1,415	15,330	10,800	7,500
Streator Dark	1.52	1.68	1.4	1,415	23,520	13,610	8,880
Chicago Common	15.3	16.5	49.1	920①	3,070	3,870	4,700

① Variation—432 to 1278.

in absorption and compressive strength of the Streator brick (light and dark) and in the modulus of rupture of the Chicago common brick.

The mortars and grout used in the construction of the beams are designated in the report as A and C mortars; consisting of 1 part cement: ¼ part hydrated lime; 3 parts sand by volume, and 1 part cement: 1 part hydrated lime: 6 parts sand by volume, respectively.

In constructing the beams, all interior joints were grouted and the mortar joints were well filled.

Regarding the design of the beams, the report states: "The beams were designed so far as possible to produce a tensile stress in the steel of 18,000 psi and a compressive strength in the brickwork of ⅓ the pier strength.

"Although a balanced design was desirable, it was impossible to obtain for all of the beams, especially types II and III where the number of bars was limited to two."

Tests of beams consisted of applying the design load and observing the initial deflection following which additional deflection due to plastic flow was observed periodically for a period of 6 months. Curves were plotted of plastic flow in inches against time in days and it was found that between the time interval of 30 to 180 days the deflection curves approximated straight lines whose equations are of the general form

$$D = b + ct$$

where D is the deflection in inches, t is the time under loading in days and b and c are constants. Constants b and c for the various beams are given in Table 2-39, reproduced from the report.

In the column headed designation, the roman numerals indicate the beam section (see Fig. 2-17). The middle letter indicates the brick (C—Chicago, S—Streator), and the final letter the mortar and grout.

TABLE 2-39

CONSTANTS FOR PLASTIC FLOW DEFLECTIONS OF BRICK BEAMS

Beam No.	Designation	Span ft.	Span in.	b in.	c in. per day
1 and 7	IV–C–A	18	9	0.070	0.00105
2 and 8	IV–S–A	21	6	0.075	0.00089
3 and 4	III–S–C	18	9	0.025	0.00167
5 and 6	III–S–A	21	6	0.055	0.00272
9 and 11	III–C–C	13	9	0.060	0.00133
10 and 12	IV–C–C	13	9	0.070	0.00044
13 and 14	IV–S–C	13	9	0.035	0.00011
15 and 16	II–C–C	8	0	0.080	0.00017
17 and 18	II–S–C	8	0	0.040	0.00105
19 and 20	II–S–A	13	9	0.170	0.00050
21 and 22	III–C–A	13	9	0.030	0.00128
23 and 24	II–C–A	13	9	0.055	0.00150

The tests reported indicate that plastic flow of reinforced brick masonry is influenced by the materials, both mortar and brick, used in the construction and, probably to a greater degree, by the number of mortar joints at right angles to the stress in the compressive section of the beam.

Comparing the plastic flow of the specimens tested with similar concrete slabs, the report states: "Comparing the deflections for the brick beams with concrete slabs of as nearly similar properties as possible, a considerable difference is found for the same periods of loading. For beams 19 and 20 (f_b = 1300 psi = ⅓ S_u, d = 3 in., L = 13 ft. 9 in.), the average deflection after 180 days was 0.255 in. For a dry cured concrete slab (f_b = 900 psi = 0.45 S_u, d = 2 5/16 in., L = 12 ft. 6 in.), the corresponding average deflection was 1.58 in."

This is a ratio of reinforced brick masonry to concrete of 1:6 and, while it might be expected that this ratio will vary for different materials, designs and spans, the conclusions of the report state: "These studies indicate that under similar conditions, the plastic flow deflection of a reinforced brick beam may be expected to be less than that of a reinforced concrete beam."

(e) **Reinforced Brick Masonry Columns.** Tests of reinforced brick masonry columns at Lehigh University by Inge Lyse are reported in the Journal of the American Ceramic Society. Vol. 16, No. 11, November 1933. These tests are described in the introduction to the report as follows: "During the early part of 1933 the Fritz Engineering Laboratory of Lehigh University tested 33 columns, the results of which are presented in this paper. All columns were about 12½ by 12½ in. in cross-section and 10 ft. in length. Five of the columns had no reinforcement, six had longitudinal steel only, nine had lateral reinforcement only, and 13 had both longitudinal and lateral reinforcement. The brick used were of three types: common solid stiff-mud brick, deaired solid brick, and deaired perforated brick. All three types of brick were of good quality; the deaired types being excellent. Five types of mortar were used: straight portland cement mortar, cement mortar containing 15 and 100 per cent lime by volume, and cement mortar containing 5 per cent celite or 15 per cent clay by weight of the cement."

The compressive strengths of the brick used in this investigation when tested flat are reported as 13,760 to 11,000 psi for the deaired solid brick, 12,190 to 7,620 psi for the deaired perforated brick, and 8,000 psi for the stiff-mud solid brick.

Table 2-40, reproduced from the report, lists the mortar mixes used and gives the ultimate strength of the columns.

Regarding the tests, the report concludes: "Both the plasticity and the strength of the mortar affected the strength of the column.

"The strength of the brick had a marked effect upon the strength of the column.

"Columns with no lateral reinforcement collapsed completely upon reaching the maximum load.

"The lateral reinforcement had little if any direct effect upon the strength of the column, but generally determined the type of failure.

"Lateral ties ¼-in. diameter in every fourth joint gave sufficient lateral reinforcement for developing the yield-point strength of the longitudinal reinforcement.

"With sufficient lateral ties, small longitudinal reinforcing bars added their full yield-point strength to the strength of the column. Large reinforcing bars added only a portion of their yield-point strength.

"The workmanship of the mason had a marked effect upon the strength of the column."

TABLE 2-40

RESULTS OF BRICK COLUMN TESTS

Mortar Mix	Admixture in mortar, %	Reinforcement		Maximum Load, lb.	Maximum Strength, psi	Type Brick (Deaired)
		Longitudinal %	Lateral			
1:3	15 lime	0	0	738,000①	4730①	Solid
1:3	15 lime	0	0	800,000	5130	Solid
1:6	100 lime	0	0	410,500	2630	Perf.
1:3	15 lime	2.0	0	800,000①	5130①	Solid
1:3	15 lime	2.0	0	810,000	5200	Solid
1:3	15 lime	2.0	0	708,700	4540	Perf.
1:6	100 lime	2.0	0	628,200	4020	Solid
1:6	100 lime	.2.0	0	473,500	3030	Perf.
1:3	15 lime	2.0	¼″ ties c	800,000①	5130①	Solid
1:3	15 lime	2.0	¼″ ties c	752,000	4820	Solid
1:3	15 lime	2.0	¼″ ties c	732,500	4700	Perf.
1:6	100 lime	2.0	¼″ ties c	483,400①	3100①	Solid
1:6	100 lime	2.0	¼″ ties c	584,200①	3740①	Solid
1:6	100 lime	2.0	¼″ ties c	671,000	4300	Solid
1:6	100 lime	2.0	¼″ ties c	527,700	3380	Perf.
1:3	0	0.67	⅜″ ties a	530,000	3400	Perf.
1:3	0	0.67	⅜″ ties b	452,800	2900	Perf.
1:3	0	0	¼″ ties b	479,300	3070	Perf.
1:3	15 clay	0	¼″ ties b	594,300	3800	Perf.
1:3	5 celite	0	¼″ ties b	531,000	3400	Perf.
1:3	5 celite	0	¼″ ties b	705,000	4520	Perf.
1:3	5 celite	0	¼″ ties b	562,000	3600	Solid②
1:3	5 celite	0	¼″ ties b	640,500	4100	③
1:3	15 clay	0	0	657,800	4220	Perf.
1:3	15 clay	0.67④	¼″ ties b	800,000	5130	Perf.
1:3	15 clay	0	¼″ ties b	489,200	3140	Perf.
1:3	15 clay	0	¼″ ties d	598,600	3830	Perf.
1:3	15 clay	0	¼″ ties a	609,200	3900	Perf.
1:2	15 clay	0	0	636,000①	4080①	Solid
1:2	15 clay	0.67④	0	690,000①	4420①	Solid
1:2	15 clay	0.67④	¼″ ties c	800,000①	5130①	Solid
1:2	15 clay	0.67④	Flats c	659,000①	4220①	Solid
1:2	15 clay	0.67	¼″ ties c	715,000①	4580①	Solid

① Tested at the age of 7 days, all others tested at 28 days.
② Underfired, low-strength brick.
③ Solid stiff-mud brick.
④ High strength steel.
a, b, c, d reinforcement in every 1st, 2nd, 3rd and 4th joint, respectively.

Professor Lyse suggests a formula for determining the strength of columns of the general form

$$P = A(f'_b + pf_s)$$

where P = ultimate strength of column
A = gross area of column
f'_b = ultimate strength of masonry
p = ratio of cross-sectional area of vertical reinforcement to gross area of column
f_s = yield-point stress of longitudinal steel.

The report recommends that f'_b be taken as the ultimate strength of masonry piers constructed of similar materials, the same type of workmanship and the same bonding arrangement as used in the full size columns, and that allowable loads on reinforced brick masonry columns be based upon the above formula with a factor of safety of 4.

Introducing this factor, the equation for allowable loads on columns becomes

$$P = A(.25\ f'_b + .25\ pf_s)$$

or, if f_s is taken as the nominal allowable stress in vertical column reinforcement of 40 per cent of the yield stress, the equation becomes

$$P = A(.25\ f'_b + .65\ pf_s).$$

Tests similar to those at Lehigh University were conducted at the University of Wisconsin and reported by Professor M. O. Withey in the paper, "Tests on Reinforced Brick Masonry Columns", ASTM Procedings, Vol. 34, Part II, 1934.

In these tests, thirty-two 12½-in. by 12½-in. brick columns 6 ft. high, constructed of 2 types of brick and 2 types of mortar, were tested. The program included the construction and test of brick masonry prisms, 8 in. by 8 in. in cross-section and 16 in. high, built of 6 courses of brick laid flat as alternate headers and stretchers with ½-in. mortar joints. The prisms were tested at the same age as the columns which was approximately 4 weeks.

Table 2-41 shows the compressive strength of mortar, brick and corresponding prisms and the modulus of elasticity of the prisms.

TABLE 2-41

PROPERTIES OF MASONRY

Column Nos.	Mortar Mix ①		Modulus of Elasticity, psi	Compressive Strength, psi		
	Volume	Weight		Mortar	Brick	Prisms
1 to 16	1:.78:4.7	1:3:12	905,000	2310	2734	1040
17 to 20	1:.78:4.7	1:3:12	2,680,000	2375	14420	3675
21 to 32	1:.78:2.9	1:3:7½	3,680,000	3567	14420	4521

The columns had varying percentages of longitudinal reinforcement ranging from none to approximately 4 per cent and varying percentages of lateral reinforcement from none to 1½ per cent. Intermediate grade 1-in. round deformed bars were used for longitudinal reinforcement, and ⅜-in.

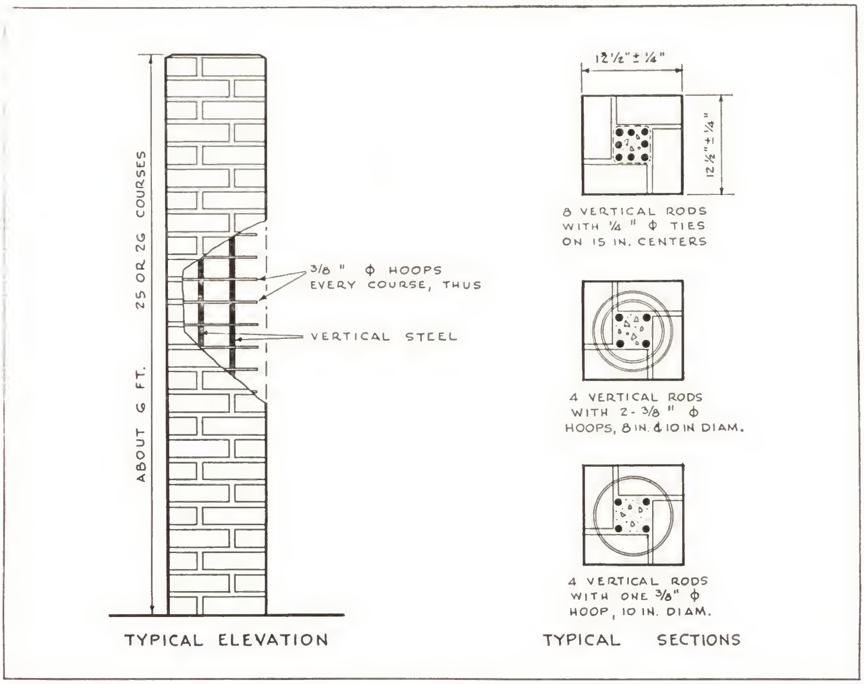


Fig. 2-18
Details of Test Columns

rods for hoops. The vertical reinforcement consisted of four or eight 1-in. rods, tied with ¼-in. rods on 15-in. centers, unless lateral reinforcement was used, in which case ⅜-in. hoops, 8 and 10 in. in diameter, were placed in every mortar joint. In some columns, a single hoop of the larger diameter was placed in each mortar bed, while in other columns both sizes of hoops were placed in every mortar bed. Fig. 2-18 shows typical elevation and sections of the columns.

Strain data indicated that the yield point of the longitudinal steel was reached in the test of every reinforced brick masonry column. Assuming the steel stress at the ultimate load on the column to be equal to the yield point, the brickwork carried the remainder of the load and the unit stress is calculated accordingly.

Table 2-42 gives a comparison of the strength of columns and prisms as well as the calculated stresses in the brickwork and steel of the columns at maximum load.

In Table 2-43, the unit stresses in the steel and brickwork were computed at one-fourth the ultimate load. The values of the secant modulus of elasticity were calculated at one-fourth the ultimate load and also after repetition of a load of 34 to 57 per cent of the maximum.

The compressive yield point of the longitudinal steel was 48,720 psi. The tensile yield point of the lateral steel was 53,800 psi.

Quoting from Professor Withey's paper: "The tests show that the effective strength of a reinforced brick masonry column equals the sum of three

TABLE 2-42

UNIT STRESSES CARRIED BY BRICKWORK AND BY STEEL IN COLUMNS

Column	Gross Area	Core Area	Percentage of Reinforcement: Longitudinal	Percentage of Reinforcement: Lateral	Maximum Load	Load at Failure of Core	Unit Stresses: Brickwork: Based on Gross Area psi.	Unit Stresses: Brickwork: Based on Core Area psi.	Unit Stresses: Steel	Ultimate Strgth. of Prisms
	sq. in.	sq. in.	%	%	lb.	lb.			psi.	psi.
No. 1	157.5	....	0	0	154800	....	983	....	....	
No. 2	160.6	....	0	0	174900	....	1090	....	....	
No. 3	162.0	....	1.92	0	270000	....	1667	....		
No. 4	161.3	....	1.93	0	255000	....	1580	....	48720	
No. 5	163.2	....	3.81	0	412300	....	2525	....		
No. 6	162.6	....	3.83	0	428800	....	2640	....		
No. 7	162.6	90.8	0	1.36	218000	190000	1340	2090	....	1040
No. 8	156.8	90.8	0	1.37	175400	143000	1120	1575	....	
No. 9	165.8	90.8	1.88	1.32	341000	323000	2060	1950		
No. 10	163.2	90.8	1.90	1.34	333500	320500	2040	1920		
No. 11	161.3	90.8	3.86	1.36	467000	378000	2900	874	48720	
No. 12	163.8	90.8	3.80	1.33	453000	....	2765	....		
No. 13	160.0	90.8	1.94	0.75	343500	....	2145	....		
No. 14	161.3	90.8	1.93	0.75	322200	269000	2000	1335		
No. 15	162.5	90.8	3.83	0.74	461750	420000	2840	1370		
No. 16	162.0	90.8	3.84	0.75	460400	389250	2840	1010		
No. 17	144.7	....	0	0	361500	....	2500	....	....	
No. 18	144.5	....	0	0	377500	....	2615	....	....	3675
No. 19	150.0	....	4.15	0	612700	....	4080	....		
No. 20	151.9	....	4.10	0	631500	....	4160	....	48720	
No. 21	148.8	90.8	0	1.54	581000	456000	3900	5030	....	
No. 22	150.7	90.8	0	1.51	564600	449000	3740	4950	....	
No. 23	150.7	90.8	2.07	1.51	608000	560500	4040	4655		
No. 24	150.7	90.8	2.07	1.52	605500	538500	4020	4410		
No. 25	152.0	90.8	4.10	1.50	768700	694000	5050	4605	48720	4521
No. 26	154.5	90.8	4.03	1.47	745000	585000	4820	3320		
No. 27	156.8	90.8	1.98	1.48	620000	515500	3950	4150		
No. 28	159.4	90.8	1.95	1.46	645000	484000	4045	3785		
No. 29	152.5	90.8	2.04	1.52	591500	510500	3880	4090		
No. 30	162.0	....	0	0	542000	....	3340	....	....	
No. 31	158.4	90.8	1.96	1.41	620000	565500	3915	4715	48720	
No. 32	154.6	90.8	2.01	0.80	712200	440500	4610	3290		

components; the strength of the plain masonry, the strength of the longitudinal steel at its yield point, and to a less certain one due to lateral restraint offered by hoops when placed in the horizontal joints. A formula involving these components which is in reasonable agreement with the results is presented.

$$\frac{P}{A} = f_b\,(1 - p) + pf_s + Kp'f'_s$$

where P = maximum load

A = gross area of cross-section

f_b = unit load for plain brick column

p = longitudinal steel ratio in terms of gross area

p' = lateral steel ratio in terms of gross area

f_s = yield point of longitudinal steel

f'_s = yield point of hoops, or weld strength if less than yield point

K = constant tentatively assumed to depend upon the ratio A/A_c and possibly on the kind of brick, where A_c = area of core within outer hoop.

TABLE 2-43

COMPUTED UNIT STRESSES AND MODULUS OF ELASTICITY

Column	Percentage of Reinforcement: Longitudinal %	Percentage of Reinforcement: Lateral %	Ultimate Strgth. psi.	Unit Stress at ¼ Ultimate Load: In Gross Area psi.	Unit Stress at ¼ Ultimate Load: In Brickwork psi.	Unit Stress at ¼ Ultimate Load: In Steel psi.	Prism Strgth. psi.	Modulus of Elasticity of Brickwork at ¼ Ultimate Load psi.: Column	Modulus of Elasticity: Column after Set	Modulus of Elasticity: Prism
No. 1......	0	0	983	246	246	....		1410000	....	
No. 2......	0	0	1090	273	273	....		1610000	1760000	
No. 3......	1.92	0	1667	417	294	6570		1910000	....	
No. 4......	1.93	0	1580	395	290	4730		2070000	....	
No. 5......	3.81	0	2525	631	365	7350		2580000	....	
No. 6......	3.83	0	2640	660	354	8340		2380000	2160000	
No. 7......	0	1.36	1340	335	335	....		1430000	1370000	
No. 8......	0	1.37	1120	280	280	....	1040	1110000	940000	905000
No. 9......	1.88	1.32	2060	515	370	8070		1920000	1860000	
No. 10.....	1.90	1.34	2040	510	356	8490		1800000	1650000	
No. 11.....	3.86	1.36	2900	725	369	9570		2270000	2240000	
No. 12.....	3.80	1.33	2765	691	336	9690		2140000	2110000	
No. 13.....	1.94	0.75	2145	536	388	8070		1990000	1990000	
No. 14.....	1.93	0.75	2000	500	370	7080		2120000	1980000	
No. 15.....	3.83	0.74	2840	710	376	9060		2350000	2220000	
No. 16.....	3.84	0.75	2840	710	294	11130		1910000	2110000	
No. 17.....	0	0	2500	625	625	....		3000000	....	
No. 18.....	0	0	2615	654	654	....		3500000	3550000	
No. 19.....	4.15	0	4080	1020	750	7260	3675	4210000	3800000	2680000
No. 20.....	4.10	0	4160	1040	779	7140		4370000	3800000	
No. 21.....	0	1.54	3800	975	975	....		3810000	3920000	
No. 22.....	0	1.51	3740	935	935	....		3520000	3680000	
No. 23.....	2.07	1.51	4040	1010	878	7200		4200000	4150000	
No. 24.....	2.07	1.52	4020	1005	804	10530		2860000	3050000	
No. 25.....	4.10	1.50	5050	1262	908	9600		3940000	4060000	
No. 26.....	4.03	1.47	4820	1205	799	10830	4521	3340000	3630000	3680000
No. 27.....	1.98	1.48	3950	987	863	7200		4100000	3730000	
No. 28.....	1.95	1.46	4045	1011	867	8200		3690000	3350000	
No. 29.....	2.04	1.52	3880	970	819	8220		3540000	....	
No. 30.....	0	0	3340	835	835	....		3270000	....	
No. 31.....	1.96	1.41	3915	979	821	8850		3310000	3600000	
No. 32.....	2.01	0.80	4610	1153	998	8640		4000000	4060000	

"For these tests with A/A_c approximately 1.7, f'_s is 54,000 psi, p' is 0.0075 for columns Nos. 1 to 16 and 0.015 for colunms Nos. 17 to 32, and K is approximately 0.6. The validity of the last term of the equation is questionable until more data on the action of hoops have been secured.

"The data indicate that with very strong brick and mortar, a ratio of gross area to core area of 1.7, 2½ per cent of lateral reinforcement and 7 per cent of longitudinal reinforcement based on core area (approximately 4 per cent based on gross area), that crushing strengths of 5000 psi may be obtained on gross area and 7000 psi on core area alone.

"The addition of ¾ per cent of hoops based on gross area promoted toughness, increased the strength and eliminated sudden failures in these tests. It seems probable that for ratios of A/A_c less than 1.5, the percentage of hooping or other desirable lateral reinforcement could be reduced to 0.6 per cent.

"With careful workmanship including filled joints and good materials, reinforced brick masonry columns should be safe under static loads with a factor of safety of 4. For columns of high-strength brick and mortar, reinforced with both longitudinal steel and hoops, a factor of safety of 3½ is probably safe.

"For common brick the strength of 8 by 8 by 16-in. masonry prisms was a good index of the strength of the plain columns, but for the high-strength brick the ratio of the plain column strength to the strength of the prisms was only 0.70 to 0.75."

Regarding the discrepancy between the prism and column strengths of "high strength" brick, the report states: "Part of these discrepancies of the Waupaca brick (high strength) columns can be ascribed to weaknesses induced by the wide vertical joints and the inferior lateral bond which existed in the columns due to the method of laying. The use of hoops in each joint of columns Nos. 21 and 22 partly overcame this deficiency and increased the ratio to 0.84 for these columns. The void spaces around the bats were another source of weakness in these columns."

As will be noted, the results of the Lehigh University and Wisconsin University tests are in close agreement and both investigators suggest a factor of safety of 4 be applied in determining the allowable load on reinforced brick masonry columns.

If the "questionable" third term in the equation proposed by Professor Withey is eliminated, the equations for allowable load suggested by both investigators become identical.

(f) Steel Columns Incased in Brick Masonry. Tests of structural steel columns incased in brick masonry are reported in Bureau of Standards Research Paper No. 520, "Compressive Strength of Steel Columns Incased in Brick Walls", by Albert L. Harris, A. H. Stang and J. W. McBurney.

Nine 6-in. H-shaped steel columns, weighing 20 lb. per ft. and 23 ft. long were tested in compression. Three of the steel columns were tested without any incasement. Six of them were incased in brick walls 14 in. thick, 6 ft. long, carried up to within approximately 8 in. of the top of the steel. The test load was applied to the top of the steel section.

The brick masonry was composed of clay brick having the following average properties: compressive strength 5253 psi, modulus of rupture 800 psi, 5-hr. boiling absorption 11.8 per cent. The mortar was composed of 1 part portland cement to 3 parts damp sand by volume, with hydrated lime added equal to 10 per cent of the cement by weight.

Three of the incased columns had the H-section oriented in one direction, while the other three had the H-section oriented perpendicular to that of the first three.

All of the incased columns failed by yielding, followed by local buckling of the short unincased portion of the steel column above the brickwork. The only failure in the brickwork was at the top where fine cracks appeared between the brick wythes, apparently caused by the local buckling of the unincased part of the steel column. Table 2-44 summarizes the data obtained from these tests.

The brick incased columns were removed from the testing machine by an overhead crane with a chain slung around the wall a few feet above the middle, as shown in Fig. 2-19. The walls were laid horizontally on the floor without a brick falling out. The fact that these walls could be handled in this manner without any indication of failure in the brick walls is evidence that the brick walls were not appreciably affected by the test loads.

The following conclusions are stated: "The brick walls effectively prevented lateral buckling of the steel column so that the maximum compressive load for the structure was, for all practical purposes, the maximum compressive

TABLE 2-44

COMPRESSIVE TESTS OF COLUMNS AND AUXILIARY SHORT LENGTHS

Column	Condition	Computed area of steel column sq. in.	Unincased length of steel column & of the short length in.	Height of Wall ft.-in.	Time between completion of walls and test days	Yield point of steel samples psi.	Compressive Strength		Load carried by	
							Column psi.	Short Length psi.	Steel Column %	Brick Wall %
1	Bare	5.90				42,800	23,900		100	0
2	Bare	5.90				47,500	23,000		100	0
3	Bare	5.85				42,000	23,100		100	0
Average						44,100	23,300		100	0
A-1	Incased	5.85	7½	22-4½	28	41,100	40,000	41,200	18	82
A-2	Incased	6.02	7½	22-4½	31	39,000	41,500	43,200	17	83
A-3	Incased	5.98	11	22-1	30	40,100	40,700	38,300	19	81
Average					30	40,100	40,700	40,900	18	82
B-1	Incased	6.03	10	22-2	29	39,000	38,800	40,600	16	84
B-2	Incased	6.14	8	22-4	31	41,400	42,100	41,500	19	81
B-3	Incased	6.10	8½	22-3½	31	40,500	41,100	42,200	16	84
Average					30	40,300	40,700	41,400	17	83

load for the portion of the steel column above the wall, which in turn was, for all practical purposes, the tensile yield point of the steel multiplied by the cross-sectional area of the steel column.

"The maximum compressive load carried by the short unincased portion of the steel columns was much greater than the maximum compressive load carried by the long unincased columns (slenderness ratio of 183) which failed by lateral buckling.

"The lateral deflection of the incased columns was negligible because in no case did it exceed 0.03 in. under the maximum load. The brick masonry showed only small cracks at the top near the steel columns.

"Over the gage length of 150 in. at mid-height of the incased columns, the steel carried, on the average, less than 20 per cent of the applied load.

"The orientation of the steel column with respect to the face of the brick wall had no effect upon the strength of the incased columns."

207. MISCELLANEOUS PROPERTIES

The heat transmission, sound resistance, fire resistance and thermal expansion of unreinforced masonry walls are discussed at considerable length in Chapter 6 of Brick and Tile Engineering, SCPI, 1950. None of these properties, with the exception of fire resistance, is materially affected by reinforcement and, consequently, data on the other properties, listed in Brick and Tile Engineering, for unreinforced walls may be applied to reinforced walls of similar dimensions.

Fig. 2-19
Steel Column Incased in Brick Masonry Being Removed from Testing Machine After Test

The fire resistance of masonry walls will be affected by the reinforcement, provided the ultimate fire resistance period is based on wall failure, in which case the reinforced walls will have higher fire resistance ratings than unreinforced walls.

The effect of reinforcement will be negligible if the ultimate fire resistance period is based on temperature rise. Consequently, the fire resistance of unreinforced masonry walls listed in Brick and Tile Engineering may be taken as minimum values for reinforced walls of similar dimensions.

The following is a brief summary of the data included in Brick and Tile Engineering on the above properties of unreinforced brick walls.

(a) Heat Transmission. Conductance and resistance coefficients of 4-in., 8-in. and 12-in. brick walls, together with surface coefficients, are given in Table 2-45. Values for "low density" and "high density" brick are taken from the ASHVE Guide and are not defined in terms of the physical properties of the brick. "Average values" are taken from National Bureau of Standards Research Paper No. 291, "Heat Transfer Through Building Walls", by M. S. Van Dusen and J. L. Fink, 1931.

TABLE 2-45

HEAT TRANSMISSION COEFFICIENTS
from ASHVE GUIDE or as indicated

Material	Transmittance (U) or Conductance (C)	Resistance (R) $(R = \frac{I}{U} = \frac{I}{C})$
Inside Surface (Still air)	(f_1) 1.65	($1/f_1$) 0.61
Outside Surface Ordinary (15 mph. wind velocity)	(f_0) 6.00	($1/f_0$) 0.17
4″ Brick (low density)	1.25	0.80
8″ Brick (low density)	0.62	1.60
8″ Brick (average value)①	0.96	1.04
12″ Brick (low density)	0.42	2.40
12″ Brick (average value)①	0.64	1.56
4″ Brick (high density)	2.30	0.44
8″ Brick (high density)	1.15	0.87
12″ Brick (high density)	0.76	1.31

① National Bureau of Standards R. P. No. 291.

(b) Sound Resistance. The sound transmission loss in decibels of 8-in., 3¾-in. and 2¼-in. brick walls plastered direct and of 2¼-in. brick walls furred and plastered are given in Table 2-46. These values are reproduced from data included in National Bureau of Standards Report BMS17, "Sound Insulation of Wall and Floor Coverings", by V. L. Chrisler.

TABLE 2-46

SOUND TRANSMISSION LOSS IN DECIBELS

Material in Test Panel	Weight psf.	Average Reduction Factors
8″ Brick—plaster both sides, brown coat and smooth white finish	97	56.7
8″ Brick—plaster both sides, brown coat and smooth white finish	87	57.2
3¾″ Brick—lime plaster with smooth lime finish on both sides—no furring	49.0	50.2
3¾″ Brick—gypsum plaster with smooth lime finish on both sides—no furring	49.0	53.7
2¼″ Brick—plaster both sides directly on brick surface	31.6	48.8
2¼″ Brick—plaster both sides on wired furring strips	36.5	53.5
2¼″ Brick—nailed furring strips and plaster both sides	38.2	55.2
2¼″ Brick—nailed furring strips and Insulite as plaster base	33.3	54.6

Since the sound reduction factor for homogeneous walls is proportional to the logarithm of the weight per unit area of the wall rather than directly proportional to the weight, increased sound insulation can usually be obtained more economically through the use of furrowing or double wall construction than by increased wall thickness.

(c) Fire Resistance. Table 2-47 gives the ultimate fire resistance periods of 4-in., 8-in. and 12-in. solid brick walls. These data are reproduced from National Bureau of Standards Report BMS92, "Fire Resistance Classifications of Building Constructions", published in 1942.

TABLE 2-47

FIRE RESISTANCE OF LOAD-BEARING BRICK WALLS

Nominal wall thickness	Type of wall	Material	Ultimate Fire Resistance Period				
			Incombustible members framed into wall or no framed-in members			Combustible members framed into wall	
			No plaster	Plaster on one side	Plaster on two sides	No plaster	Plaster on exposed side
in.			hr.	hr.	hr.	hr.	hr.
4	Solid	Clay or shale	1¼	1¾	2½	......	......
8	Solid	Clay or shale	5	6	7	2	2½
12	Solid	Clay or shale	10①	10①	12①	8	9

① Based on load failure. If based on temperature rise, the fire resistance period would be 12 hr. for the unplastered wall, 13 hr. for plaster on one side, and 15 hr. for plaster on both sides.

Note: Not less than ½-in. 1:3 sanded gypsum plaster is required to develop the above ratings for plastered walls.

As previously indicated, ultimate fire resistance periods for unreinforced brick walls may be considered minimum values for reinforced brick masonry.

(d) Thermal Expansion. The coefficient of thermal expansion of brick masonry varies, depending upon the coefficients of both the brick and the mortar. An average value may be taken as 0.000004 in. per in. per degree F. The expansion of reinforced brick masonry will obviously be affected by the amount and location of the reinforcement.

CHAPTER 3

MINIMUM DESIGN AND CONSTRUCTION REQUIREMENTS

300. INTRODUCTION

The recommendations in this chapter regarding the design and construction of reinforced masonry are presented in the form of Building Code Requirements. Various drafts of these regulations have been reviewed by the Engineering Subcommittee of the Pacific Coast Building Officials Conference and by the Drafting Subcommittee of American Standard Association's Sectional Committee A41 on Building Code Requirements and Good Practice Recommendations for Masonry with a view to developing a national standard. Such a standard is still under consideration.

While the recommendations presented in this chapter are those of the authors, they are glad to acknowledge the constructive criticism and helpful suggestions of these committees which aided them materially in developing the regulations.

301. GENERAL

(a) Scope. These regulations cover the use of reinforced masonry in any structure to be erected under the provisions of this building code of which they form a part. They are intended to supplement the provisions of the general code in order to provide for the proper design and construction of structures of reinforced masonry. In all matters pertaining to design and construction where these specific regulations are in conflict with other portions of the code, these regulations shall govern.

(b) Administration.

1. Drawings and typical details of all reinforced masonry construction showing the sizes and position of all structural members, steel reinforcement, design strengths, live loads and lateral forces used in the design shall be filed with the building department before a permit to construct such work shall be issued. Calculations pertaining to the design shall be submitted, if requested by the building official.

2. The sponsors of any system of reinforced masonry which has been in successful use or the adequacy of which has been shown by test and the design of which is either in conflict with or not covered by these regulations shall have the right to present the data on which their design is based in accordance with the provisions of the building code regarding alternate materials and constructions.

3. Satisfactory assurances shall be given the building official that sufficient supervision will be provided to obtain the required quality of workmanship. If the building official is not satisfied that the extent of supervision and inspection provided will assure such workmanship, he may require that the allowable working stresses used in design be reduced.

(c) Definitions. The following terms are defined for use in this code:

BONDER—A header or other masonry unit overlapping and bearing upon two or more leaves, tiers or wythes.

COLUMN—A compression member, vertical or nearly vertical, the height of which exceeds four times its least lateral dimension.

CONCRETE—A mixture of portland cement, fine aggregate, coarse aggregate and water.

CORBEL—That part of a structure built outward from the face by projecting courses of masonry.

DEFORMED BAR—A reinforcing bar conforming to the Standard Specifications for Minimum Requirements for Deformations of Deformed Steel Bars for Concrete Reinforcement (ASTM Designation A305-). Bars not conforming to these specifications are classed as plain bars.

EFFECTIVE AREA OF REINFORCEMENT—The area obtained by multiplying the right cross-sectional area of the metal reinforcement by the cosine of the angle between its direction and the direction for which the effectiveness of the reinforcement is to be determined.

GROUT—Mortar to which sufficient water is added to produce pouring consistency without segregation of the constituents of the mortar.

HEADER—A masonry unit laid lengthwise across a wall.

HOLLOW MASONRY—Masonry consisting wholly or in part of hollow masonry units laid contiguously in mortar or grout.

MASONRY—A built-up construction or combination of masonry units set in mortar and grout.

MASONRY UNIT—Any brick, tile, or block conforming to the requirements of Section 302(b).

MORTAR—A plastic mixture of cementitious material, fine aggregate and water.

PARTIALLY REINFORCED MASONRY—Masonry in which reinforcement is provided to carry the principal tensile stresses but which does not conform to the requirements for reinforced masonry as set forth in this code.

REINFORCED MASONRY—Masonry in which reinforcement is embedded as required in these regulations and in such a manner that the two materials act together in resisting forces.

REINFORCEMENT—Structural steel shapes, steel bars, rods, wire mesh or expanded metal embedded or incased in masonry to increase the resistance of the masonry to internal stresses.

SOLID MASONRY—Masonry consisting wholly of solid masonry units laid contiguously in mortar or grout.

STRETCHER—A unit laid with its length horizontal and parallel with the face of the wall or other masonry member.

302. MATERIALS AND TESTS

(a) Tests.

1. The building official, or his authorized representative, shall have the right to order the test of any material entering into reinforced masonry, when there is reasonable doubt as to its suitability for the purpose; to order reasonable tests of the masonry from time to time to determine whether the materials and workmanship are such as to produce reinforced masonry of the necessary quality; and to order the test under load of any portion of a completed structure, when the conditions have been such as to

leave reasonable doubt as to the adequacy of the structure to serve the purpose for which it is intended.

2. Tests of materials shall be made in accordance with the standard method prescribed for the material in question.

3. When a load test is required, the member or portion of the structure under consideration shall be subjected to a superimposed load equal to twice the design live load plus one-half of the dead load. This load shall be left in position for a period of 24 hr. before removal. If, during the test, or upon removal of the load, the member or portion of the structure shows evidence of failure, such changes or modifications as are necessary to make the structure adequate for the rated capacity shall be made; or where lawful, a lower rating shall be established.

The maximum deflection, D, of a flexural member at the end of the 24-hr. period shall exceed neither L/200 nor $\frac{L^2}{4000t}$ in which

D = Maximum deflection of the member, inches
L = Span of the member, inches
t = Total thickness or depth of the member, inches

If the maximum deflection exceeds 0.10 in., the residual deflection, within 24 hr. after the removal of the load, shall not exceed 30 per cent of the maximum deflection. However, if the residual deflection exceeds 30 per cent, the member may be retested if this is approved by the building official. The flexural member shall be considered to conform to the load test requirements if, in the second test, the residual deflection does not exceed 10 per cent of the maximum deflection caused by the second loading.

(b) Materials.

1. Brick. Clay or shale brick shall conform to the requirements of Standard Specifications for Building Brick, ASTM C62-.

Concrete brick shall conform to the requirements of Grade A units of Standard Specifications for Concrete Building Brick, ASTM C55-.

Sand-lime brick shall conform to the requirements of Standard Specifications for Sand-Lime Brick, ASTM C73-.

2. Structural Clay Tile. Structural clay tile if used for load-bearing purposes shall conform to American Standard Specifications for Structural Clay Load-Bearing Wall Tile (ASTM C34–; ASA A74.1–.)

Structural clay tile used for non-load-bearing purposes shall conform to American Standard Specifications for Structural Clay Non-Load-Bearing Tile (ASTM C56–; ASA A76.1–.)

3. Concrete Masonry Units. Load-bearing concrete masonry units shall conform to American Standard Specifications for Hollow Load-Bearing Concrete Masonry Units (ASTM C90–; ASA A79.1–) or American Standard Specifications for Solid Load-Bearing Concrete Masonry Units (ASTM C145–; ASA A81.1–).

Non-load-bearing concrete masonry units shall conform to American Standard Specifications for Hollow Non-Load-Bearing Concrete Masonry Units (ASTM C129–; ASA A80.1–).

4. Reinforcement. Reinforcement for reinforced masonry shall conform to the following applicable standards; deformed reinforcing bars also shall conform to the requirements of Standard Specifications for Minimum Require-

ments for the Deformations of Deformed Steel Bars for Concrete Reinforcement, ASTM A305–.

American Standard Specifications for Billet-Steel Bars for Concrete Reinforcement (ASTM A15–; ASA A50.1–).

American Standard Specifications for Rail-Steel Bars for Concrete Reinforcement (ASTM A16–; ASA A50.2–).

American Standard Specfications for Axle-Steel Bars for Concrete Reinforcement (ASTM A160–; ASA G43.1–).

American Standard Specifications for Cold-Drawn Steel Wire for Concrete Reinforcement (ASTM A82–; ASA A50.3–).

American Standard Specifications for Welded Steel Wire Fabric for Concrete Reinforcement (ASTM A185–; ASA G45.1–).

American Standard Specifications for Steel for Bridges and Buildings (ASTM A7–; ASA G24.1–).

5. Mortar. Mortar and grout for reinforced masonry shall consist of portland cement, hydrated lime or lime putty and aggregate and shall conform to the requirements for Type A-1 or Type A-2 mortar of Tentative Specifications for Mortar for Unit Masonry, (ASTM C270–). Coarse aggregate conforming to the requirements of Standard Specifications for Concrete Aggregate (ASTM C33–) may be added to the mortar or grout as provided in Section 304(g).

303. WORKING STRESSES AND TESTS

Notation

d = Depth from compression face of beam or slab to center of longitudinal tensile reinforcement; the least lateral dimension of a column or prism.

h = Unsupported length of column or prism.

f_m = Compressive unit stress in extreme fiber of masonry in flexure.

f'_m = Compressive strength of masonry at age of 28 days unless otherwise specified.

f_s = Tensile unit stress in main reinforcement; nominal working stress in vertical column reinforcement.

f_v = Tensile unit stress in web reinforcement.

n = Ratio of modulus of elasticity of steel to that of masonry.

u = Bond stress per unit of surface area of bar.

v_m = Shearing unit stress permitted in masonry.

v = Shearing unit stress.

E_m = Modulus of elasticity of masonry.

E_s = Modulus of elasticity of reinforcement.

(a) Masonry Strength. For the design of reinforced structures, the value of f'_m used for determining the allowable stresses as stipulated in Section 303(d) shall be based on the specified minimum 28-day compressive strength of the masonry or on the specified minimum compressive strength at the earlier age at which the masonry may be expected to receive its full load. All plans, submitted for approval or used on the job, shall clearly show the assumed strength of masonry at a specified age for which all parts of the structure were designed.

(b) Determination of Masonry Strength.

1. The determination of the compressive strength of solid masonry (f'_m) shall be made by one of the following methods:

Method No. 1. When the strength of the solid masonry is to be established by preliminary tests, the tests shall be made in advance of the operations using prisms built of similar materials under the same conditions and, insofar as possible, with the same bonding arrangement as for the structure. In building the prisms, the moisture content of the unit at time of laying, the consistency of the mortar and the workmanship shall be the same as will be used in the structure. The test prisms for beams and slabs shall be built of representative units, with their long dimension horizontal.

Unless permission is otherwise given, all specimens shall have a height-to-thickness ratio (h/d) not less than 2 and shall be not less than 16 in. in height. If h/d differs from 2, the value of f'_m shall be taken as the compressive strength of the specimens multiplied by a correction factor as follows:

Ratio of height to thickness (h/d)	1.5	2.0	2.5	3.0	4.0	5.0	6.0
Correction factor	0.86	1.00	1.11	1.20	1.33	1.43	1.50

Factors between those listed shall be determined by direct interpolation.

Method No. 2. When the strength of the solid masonry is not determined by preliminary tests and the units, mortar and workmanship conform to all applicable requirements of these regulations, the allowable stresses may be based on an assumed value f'_m not exceeding 60 per cent of the compressive strength of the units (psi) when Type A-1 mortar is used and not exceeding 45 per cent of the compressive strength when Type A-2 mortar is used, but such assumed value of f'_m shall not exceed 2000 psi nor 1500 psi respectively. Masonry units shall be tested in accordance with standard methods as prescribed in the specifications for the units.

2. The determination of the compressive strength of hollow masonry (f'_m) shall be made by one of the following methods:

Method No. 1. When the strength of the hollow masonry is to be established by preliminary tests, the tests shall be made in advance of the operations, using prisms built of similar materials, under the same conditions and, insofar as possible, with the same bonding arrangement as for the structure.

Test prisms shall be built in the form of hollow squares 8 by 8 in. in plan and 16 in. high or in the form of rectangles 8 by 16 in. in plan and 16 in. high. The hollow core shall not be filled with grout.

Masonry strength (f'_m) shall be computed by dividing the ultimate load by the net area of the masonry units used in construction of the prisms.

Method No. 2. When the strength of the hollow masonry is not determined by preliminary tests and the units, mortar and workmanship conform to all applicable requirements of these regulations, the allowable stresses may be based on an assumed value f'_m not exceeding 60 per cent of the net compressive strength of the units (psi) when Type A-1 mortar is used and not exceeding 45 per cent of the net compressive strength when Type A-2 mortar is used, but such assumed value f'_m shall not exceed a value of 1500 psi nor 1000 psi, respectively. Masonry units shall be tested in accordance with standard methods as prescribed in the specifications for the units.

(c) Tests of Masonry Prisms.

1. Test prisms shall be constructed as prescribed in Section 303(b). The prisms shall be stored damp for three days at a temperature not less than 65° F. and then in air at a temperature not less than 65° F. The ends of each prism shall be capped with a suitable material such as calcined gypsum to provide

bearing surfaces plane within 0.003 in. and approximately perpendicular to the axis of the prism. The prism shall then be tested in accordance with the relevant provisions of the Standard Method for Compressive Strength of Molded Concrete Cylinders, ASTM C39-.

2. Not less than three specimens shall be made for each test.

3. The standard age of test specimens shall be 28 days, but 7-day tests may be used, provided the relation between the 7- and 28-day strengths of the masonry is established by test for the materials used.

(d) Allowable Stresses in Reinforced Masonry. The allowable stresses in reinforced masonry shall not exceed the values shown in the following table:

	Stress	Allowable Stress psi for any strength of f'_m	
f_m	Compression—axial	①	
f_m	Compression—flexural	$0.33\ f'_m$	
v_m	Shear—no web reinforcement	50 psi	
v	Shear—with web reinforcement	150 psi	
f_m	Bearing	$0.25\ f'_m$	
E_m	Modulus of elasticity	$1000\ f'_m$	
E_v	Modulus of rigidity in shear	$400\ f'_m$	
	Bond	Mortar or grout	
		A–1	A–2
		psi	psi
u	Plain bars	80	60
u	Deformed bars (ASTM A305–)	160	120

① See Sections 308(c) and 308(h).

(e) Allowable Stress in Reinforcement. Unless otherwise provided in this code, the limiting stresses (f_s) in reinforcing steel shall not exceed the following:

f_s tensile unit stress in longitudinal reinforcement and f_v tensile unit stress in web reinforcement:

- Structural grade steel bars .... $f_s = 18{,}000$ psi
- Structural steel shapes .... $f_s = 18{,}000$ psi
- Intermediate grade steel bars, and hard-grade bars (billet steel, rail steel, or axle steel) .... $f_s = 20{,}000$ psi

f_s compression in column verticals:

- Intermediate grade steel bars .... $f_s = 16{,}000$ psi
- Hard-grade steel bars (billet steel, rail steel, or axle steel) .... $f_s = 20{,}000$ psi

304. MASONRY CONSTRUCTION

(a) Preparation of Equipment and Place of Construction. Before placing masonry, all forms and equipment shall be cleaned, all debris and ice shall be removed from the spaces to be occupied by the masonry, and the reinforcement shall be thoroughly cleaned of ice or other coatings.

(b) Preparation of Masonry Units.

1. At the time of laying, all units shall be sound and clean.

2. *Brick Made From Clay or Shale.* At the time of laying, all units shall have sufficient moisture content so that the amount of water absorbed during the suction test, as described in ASTM Standard Methods of Sampling and Testing Brick, C67-, is not more than 0.025 oz. per sq. in. per min.

3. *Concrete Masonry Units.* Concrete masonry units shall not be wetted before laying in the wall except in extremely dry desert areas where the bearing surfaces of the face shells may be slightly moistened immediately before laying to prevent excessive suction.

(c) Mixing Mortar and Grout.

1. Mortar and grout shall be mixed in a mechanically operated batch mixer except as provided in Section 304(c)4. The mixer shall thoroughly intermingle all ingredients by cutting, sliding and rubbing of the mass within itself. The mortar or grout shall be mixed for a period of at least 3 min. after all materials for a batch are in the drum. The drum must be completely emptied before the succeeding batch of materials is placed therein.

2. Grout shall consist of mortar to which sufficient water has been added to produce a pouring consistency and it shall be stirred or worked at frequent intervals before placing to prevent separation of the materials.

3. The consistency of mortar shall be adjusted to the satisfaction of the mason but as much water shall be added as is compatible with convenience in using the mortar. If the mortar begins to stiffen from evaporation or absorption of a part of the mixing water, the mortar shall be retempered by adding water and remixing. Mortar or grout shall not be used after it has begun to set.

4. Hand mixing will be permitted on small jobs, provided the quantity of the materials is controlled and accurately maintained and, provided further, that the hand mixing is done in a manner meeting with the approval of the building official.

(d) Bonded Masonry.

1. Brick shall be laid plumb with full mortar joints. Mortar beds shall be spread smooth. The ends of the brick shall be buttered with sufficient mortar to completely fill the end joint when the brick is placed, and the vertical longitudinal joints shall be completely filled. Vertical longitudinal (collar) joints shall be filled by parging the facing or back-up, by pouring the vertical joint full of grout, by the pick and dip method, or by shoving. Closures may be rocked into place with head joints thrown against the two adjacent brick in place.

2. Facing and backing shall be bonded so that not less than 4 per cent of the wall surface of each face is composed of bonders (headers) extending not less than 4 in. into the backing. The distance between the adjacent bonders shall not exceed 24 in. either vertically or horizontally.

(e) Grouted Masonry. Brick shall be laid plumb. All brick in the outer tiers shall be laid with full head and bed joints and all interior joints shall be filled with grout. No mortar shall be placed or allowed to remain in the grout space. Mortar which protrudes from bed or head joints shall be removed before pouring the grout. Brick in the interior tiers shall be placed or floated in grout poured between the two outer tiers. One of the outer tiers may be carried up not more than three courses before grouting but the other shall be carried up not more than one course above the grout. Each pour of grout shall be stopped at least 1½ in. below the top and properly puddled. The longitudinal vertical joints shall be not less than ¾ in. wide. Bonding headers shall not be used.

(f) Hollow Unit Masonry.

1. All units shall be laid plumb with full face shell mortar beds. All head (or end) joints shall be filled solidly with mortar for a distance in from the face of the unit or wall not less than the thickness of the longitudinal

face shells. Cross webs adjacent to vertical cores which are to be filled with grout shall be fully bedded in mortar to prevent leakage of grout.

2. Bond in hollow unit masonry shall be provided by lapping units in successive vertical courses or by equivalent mechanical anchorage.

3. All hollow unit masonry shall be built to preserve the unobstructed vertical continuity of the cores to be filled. Mortar "fins" protruding from joints shall be removed before pouring grout. The minimum continuous clear dimensions of vertical cores shall be 2 in. In filling vertical cores, the grout pour shall not exceed 4 ft. in height unless cleanouts are left open at the bottom masonry course of each core to be reinforced and such cleanouts closed only after inspection of the core space and the setting of the vertical reinforcement in fixed position.

4. Grout shall be poured in lifts of less than 4 ft., allowing at least 15 min. for settlement of grout before pouring the next lift. Grout shall be rodded or puddled during placement to insure complete filling of the core. When grouting is stopped for 1 hr. or longer, the grout pour shall be stopped 1½ in. below the top of a masonry unit.

5. Horizontal beams may be built of hollow unit masonry, using channelled units to permit horizontal reinforcement to be placed in the desired position. The top of unfilled cores below such horizontal beams shall be covered to confine the grout fill to the beam section. No material shall be used which destroys the bond between courses. Grouting of beams over openings shall be done in a continuous operation. All grout shall be puddled in place to insure complete filling of cores and incasement of reinforcement.

(g) Pea Gravel Grout. In grout spaces in brick masonry 2 in. or more in both horizontal dimensions and in grout spaces in filled cell construction 4 in. or more in both horizontal dimensions, the grout may contain an addition of pea gravel equal to not more than 2 parts by volume of cement used. Such pea gravel shall be graded with not more than 5 per cent passing the No. 8 sieve and with not less than 95 per cent passing the ⅜-in. sieve. Brick pieces or chips may be embedded into grout in such spaces provided each piece or chip is surrounded by not less than ½ in. of grout. Where the minimum continuous clear opening of a grout space in filled cell construction exceeds 8 in., it may be filled and treated as reinforced concrete.

(h) Joints. Exterior joints shall be trowel-pointed or shall be tooled with a jointer just before the initial set occurs, compacting the mortar into the joint and against the masonry units with firm pressure.

(i) Cold Weather Requirements.

1. Adequate equipment shall be provided for heating the masonry materials and protecting the masonry during freezing or near-freezing weather. No frozen materials or materials containing ice shall be used.

2. Sand shall be heated in such a manner as to remove frost or ice. Water or sand shall not be heated to a temperature above 160° F. When necessary to remove frost, the masonry units shall be heated.

3. Whenever the temperature of the surrounding air is below 40° F., all newly constructed reinforced masonry laid in mortar, in which high-early-strength portland cement is used, shall be maintained at a temperature of at least 50° F. for not less than 24 hr. by means of enclosures, artificial heat or by other protective methods as will meet the approval of the building official. When any cementing material other than high-early-strength portland cement is used, these temperatures shall be maintained for at least 72 hr.

4. All methods and materials for the protection of the fresh masonry work against freezing shall be subject to the approval of the building official. In general, the methods and materials now commonly accepted as suitable for the protection of reinforced concrete construction in freezing weather shall be used. Salt or other chemicals for lowering the freezing temperature of the mortar shall not be used.

305. DETAILS OF CONSTRUCTION

(a) Design of Forms. Forms shall conform to the shape, lines and dimensions of the members as called for on the plans, and shall be substantial and sufficiently tight to prevent leakage of mortar. They shall be properly braced or tied together so as to maintain position and shape.

(b) Removal of Shores and Forms.

1. In no case shall shores and forms be removed until it is certain that the masonry has hardened sufficiently to carry its own weight and all other reasonable temporary loads that may be placed on it during construction. The results of suitable control tests may be used as evidence that the masonry has attained such sufficient strength.

2. For girders and beams, the minimum time which shall elapse before removal of shores or forms shall be 10 days after the completion of the member, providing that suitable curing conditions have been obtained during that period. The forms and shores under slabs shall not be removed in less than 7 days after completion of such slabs and then only when suitable curing conditions have obtained throughout the entire curing period. At least 16 hr. shall elapse after building masonry columns or walls before constructing floor or roof applied as a uniform load and an additional 48 hr. shall elapse before applying a concentrated load, such as truss, girder or beam.

(c) Pipe, Conduit, etc., Embedded in Masonry. Pipes which will contain liquid, gas or vapor at other than room temperature shall not be embedded in masonry necessary for structural stability or fire protection. Drain pipes and pipes whose contents will be under pressure greater than atmospheric pressure by more than 5 psi shall not be embedded in structural masonry except in passing through from one side to the other of a floor, wall or beam. Placement of pipe or conduits in unfilled cores of hollow unit masonry shall not be considered as embedment. Electric conduits and other pipes whose embedment is allowed shall not, with their fittings, displace that masonry of a column on which stress is calculated or which is required for fire protection, to greater extent than 4 per cent of the area of the cross-section. Sleeves or other pipes passing through floors, walls or beams shall not be of such size or in such location as unduly to impair the strength of the construction; such sleeves or pipes may be considered as replacing structurally the displaced masonry, provided they are not exposed to rusting or other deterioration, are of iron or steel not thinner than standard wrought-iron pipe, have a nominal inside diameter not over 2 in., and are spaced not less than 3 diameters on center. Embedded pipes or conduits other than those merely passing through shall not be larger in outside diameter than ⅓ the thickness of the slab, wall or beam in which they are embedded; shall not be spaced closer than 3 diameters on centers, nor so located as unduly to impair the strength of the construction.

(d) Cleaning and Bending Reinforcement. Before being placed, all metal reinforcement shall be free from loose rust and other coatings that will destroy or reduce the bond. All reinforcement shall be accurately cut to length and bent by such methods as will prevent injury to the material. All

kinks or bends in the bars caused by handling incident to delivery shall be straightened out without injury to the material before placing it in the masonry.

(e) Placing Reinforcement.

1. The minimum clear distance between parallel bars except in columns shall be equal to the nominal diameter of the bar.

2. In solid unit masonry, vertical reinforcement shall be accurately placed and held rigidly in position before work is started. Horizontal reinforcement may be placed as the work progresses.

3. In hollow unit masonry, vertical reinforcement may be placed after cleaning cores ready for inspection and before inspection occurs. Vertical reinforcement shall be accurately placed and shall be held in position at intervals not exceeding 160 times the minimum dimension of the reinforcement. Horizontal reinforcement may be placed as the work progresses.

(f) Splices in Reinforcement. Splices may be made only at such points and in such manner that the structural strength of the member will not be reduced. Lapped splices shall provide sufficient lap to transfer the working stress of the bars by bond and shear. Welded or mechanical connections shall develop the design strength of the bar.

(g) Protection for Reinforcement. All bars shall be completely embedded in mortar. All reinforcing steel shall have a coverage of masonry not less than the following:

1. Three inches for bottoms of footings or vertical members exposed to soil.

2. Two inches on vertical members where masonry is exposed to action of weather.

3. One and one-half inches for all reinforcement in columns.

4. One and one-half inches on the bottom and sides of beams or girders.

5. Three-fourths inch from the faces of all walls not exposed to action of weather or soil.

6. One bar diameter over all bars not less than ¾ in. at the upper faces on any member except where exposed to weather or soil in which cases the minimum coverage shall be 2 in. or 3 in., respectively.

(h) Minimum Joint Thickness. The thickness of grout or mortar between masonry units and reinforcement shall be not less than ¼ in. except that ¼-in. bars may be laid in ½-in. horizontal mortar joints. Vertical joints containing both horizontal and vertical reinforcement shall be not less than ½ in. larger than the sum of the diameters of the horizontal and vertical reinforcement contained therein.

(i) Construction Joints. Where fresh masonry joins masonry that is partially set or totally set, the exposed surface of the finished masonry shall be cleaned with a wire brush and dampened when necessary to obtain the best possible bond with the new work. All loose masonry units and mortar shall be removed.

(j) Chases and Recesses. Chases and recesses in masonry walls shall not be constructed so as to reduce the required strength, thickness, or fire resistance of the wall.

(k) Arches and Lintels. The heads of openings which support masonry shall be lintels of incombustible materials or masonry arches.

306. DESIGN—GENERAL CONSIDERATIONS

Notations

f'_m = Compressive strength of masonry at age of 28 days; unless otherwise specified.

n = Ratio of modulus of elasticity of steel to that of masonry = $\frac{E_s}{E_m}$;

assumed as equal to $\frac{30{,}000}{f'_m}$

w = Uniformly distributed load per unit of length of beam or slab.

l' = Clear span for positive moment and the average of the two adjacent clear spans for negative moment.

(a) Assumptions. The design of reinforced masonry shall be in accordance with following principal assumptions:

1. A section that is plane before bending remains plane after bending.
2. Moduli of elasticity of the masonry and of the reinforcement remain constant.
3. Tensile forces are resisted only by the tensile reinforcement.
4. Reinforcement is completely surrounded by and bonded to masonry.

(b) Design Loads. The provisions for design herein specified are based on the assumption that all structures shall be designed for all dead and live loads coming upon them; the live loads to be in accordance with the general requirements of the building code of which this forms a part, with such reductions for structural members as are permitted therein.

(c) Resistance to Wind, Blast and Earthquake Forces.

1. The moments, shears and direct stresses resulting from wind, blast or earthquake forces determined in accordance with recognized methods shall be added to the maximum stresses which obtain at any section for dead and live loads.

2. For stresses due to wind, blast or earthquake combined with dead and live loads, the allowable stresses in this chapter may be increased 33⅓ per cent, provided the strength of the section thus formed is not less than that required for dead and live loads alone.

3. Wind, blast, and earthquake stresses may be assumed to never occur at the same time.

(d) Flexural Computations.

1. All members shall be designed to resist at all sections the maximum bending moment and shears produced by dead load, live load and other forces, as determined by the principles of continuity and relative rigidity.

2. In the case of two or more spans where the larger of the two adjacent spans does not exceed the shorter by more than 20 per cent and loads are uniformly distributed and the unit live load does not exceed 3 times the unit dead load, the following moments and shears may be used:

Negative moment at exterior face of first interior support:

For beams and girders:

Two spans.......................... $\frac{1}{8}\,wl'^2$

More than two spans.................. $\frac{1}{10}\,wl'^2$

Negative moment at face of other interior supports: $\frac{1}{12}$ wl'^2

Positive moment at center of span:

End spans.................................. $\frac{1}{10}$ wl'^2

Interior spans.............................. $\frac{1}{12}$ wl'^2

Shear in end members at first interior support 1.20 $\frac{wl'}{2}$

Shear at other supports........................ $\frac{wl'}{2}$

3. The span length of freely supported beams shall be the clear span plus the depth of beam, but shall not exceed the distance between centers of the supports.

4. In the application of the principle of continuity, center to center distances may be used in the basic moment determination of all members. Moments actually prevailing at the faces of supports shall be used for the design of beams and girders at such points.

5. The depth of the beam or slab shall be taken as the distance from the centroid of the tensile reinforcement to the compression face.

6. The clear distance between lateral supports of a beam shall not exceed 32 times the least width of compression flange.

7. Compression steel in beams or girders shall be anchored by ties or stirrups not less than ¼ in. in diameter, spaced not farther apart than 16 bar diameters or 48 tie diameters. Such ties or stirrups shall be used throughout the distance where compression steel is required.

The effectiveness of compression reinforcement in resisting bending may be taken at twice the value indicated from calculations assuming a straight line relation between stress and strain and the modular ratio given in "Notation" at the beginning of this chapter, but not of greater value than the allowable stress in tension.

(e) Combined Axial and Flexural Stresses. Members subject to combined axial and flexural stresses shall be so proportioned that the quantity

$$\frac{f_a}{F_a} + \frac{f_m}{F_m} \text{ shall not exceed 1}$$

Where f_a = Computed axial unit stress = Total axial load/area.

F_a = Axial unit stress permitted by this code at point under consideration if member were carrying axial load only, including any increase in stress allowed by Section 306 (c).

f_m = Computed flexural unit stress.

F_m = Flexural unit stress permitted by this code, if member were carrying bending load only, including any increase in stress allowed by Section 306 (c).

307. SHEAR, DIAGONAL TENSION, BOND AND ANCHORAGE

Notation

A_v = Total area of web reinforcement in tension within a distance of s (measured in a direction parallel to that of the main reinforcement), or the total area of all bars bent up in any one plane.

a = Angle between inclined web bars and axis of beam.

b = Width of rectangular flexural member or width of flange for T and I sections.

d = Depth from compression face of beam to centroid of longitudinal tensile reinforcement.

f'_m = Compressive strength of masonry at age of 28 days unless otherwise specified.

f_v = Tensile stress in web reinforcement.

j = Ratio of distance between centroid of compression and centroid of tension to the depth d.

s = Spacing of stirrups or of bent bars in a direction parallel to that of the main reinforcement.

u = Bond stress per unit of surface area of bar.

v = Shearing stress.

V = Total shear.

V′ = Excess of the total shear over that permitted on the masonry.

Σo = Sum of perimeters of bars in one set.

(a) Shearing Unit Stress.

1. The shearing stress v, as a measure of diagonal tension, in reinforced masonry flexural members shall be computed by the following formula:

$$v = \frac{V}{bjd}$$

2. When the value of the computed shearing stress exceeds the shearing unit stress v_m permitted on the masonry of an unreinforced web, web reinforcement shall be provided to carry the excess.

(b) Types of Web Reinforcement.

1. Web reinforcement may consist of:

(1) Stirrups or web reinforcement bars perpendicular to the longitudinal steel.

(2) Stirrups or web reinforcement bars welded or otherwise rigidly attached to the longitudinal steel and making an angle of 30° or more thereto.

(3) Longitudinal bars bent so that the axis of the inclined portion of the bar makes an angle of 15° or more with the axis of the longitudinal portion of the bar.

(4) Special arrangements of bars with adequate provisions to prevent slip of bars or splitting of the masonry by the reinforcement.

2. Stirrups or other bars to be considered effective as web reinforcement shall be anchored at both ends.

(c) Stirrups.

1. The area of steel required in stirrups placed perpendicular to the longitudinal reinforcement shall be computed by the following formula:

$$A_v = \frac{V's}{f_v jd}$$

2. Inclined stirrups shall be proportioned by the formula in Section 307(d)3.

(d) Bent Bars.

1. When the web reinforcement consists of a single bent bar or of a single group of bent bars, the required area of such bars shall be computed by the following formula, provided V′ shall not exceed 0.040 $f'_m bjd$:

$$A_v = \frac{V'}{f_v \sin a}$$

2. Only the center ¾ of the inclined portion of such bar, or group of bars, shall be considered effective as web reinforcement.

3. Where there is a series of parallel bent bars, the required area shall be determined by the following formula:

$$A_v = \frac{V's}{f_v jd(\sin a + \cos a)}$$

(e) Spacing of Web Reinforcement. Where web reinforcement is required, it shall be so spaced that every 45° line (representing a potential crack) extending from the mid-depth of the beam to the longitudinal tension bars shall be crossed by at least one line of web reinforcement.

(f) Computation of Bond Stress in Beams. In flexural members in which tensile reinforcement is parallel to the compressive face, the bond stress u shall be computed by the formula:

$$u = \frac{V}{\Sigma o\ jd}$$

in which V = total external shear at the section.

(g) Anchorage Requirements.

1. Tensile negative reinforcement in any span of a continuous, restrained or cantilever beam, or in any member of a rigid frame, shall be adequately anchored by bond, hooks or mechanical anchors in or through the supporting member. Within any such span every reinforcing bar, whether required for positive or negative reinforcement, shall be extended at least 12 diameters beyond the point at which it is no longer needed to resist stress. The maximum tension in any bar must be developed by bond on a sufficient straight or bent embedment or by other anchorage. If preferred, the bar may be bent across the web at an angle of not less than 15° with the longitudinal portion of the bar and made continuous with the reinforcement which resists moment of opposite sign.

2. Of the positive reinforcement in continuous beams, not less than ¼ the area shall extend along the same face of the beam into the support a distance of 6 in.

3. In simple beams, or at the freely supported end of continuous beams, at least ⅓ the required positive reinforcement shall extend along the same face of the beam into the support a distance of 6 in.

4. Plain bars in tension shall terminate in standard hooks except that hooks shall not be required on the positive reinforcement at interior supports of continuous members.

(h) Anchorage of Web Reinforcement.

1. Single separate bars used as web reinforcement shall be anchored at each end by one of the following methods:

(1) Welding to longitudinal reinforcement.

(2) Hooking tightly around the longitudinal reinforcement through 180°.

(3) Embedment above or below the mid-depth of the beam on the compression side a distance sufficient to develop the stress to which the bar will be subject at a bond stress of not to exceed 80 psi on plain bars nor 160 psi on deformed bars.

(4) Standard hook (see Section 307(i)1, considered as developing 7500 psi, plus embedment sufficient to develop by bond the remainder of the stress to which the bar is subjected. The unit bond stress shall not exceed that specified in Section 303(d). The effective embedded length shall not be assumed to exceed the distance between the mid-depth of the beam and the tangent of the hook.

2. The extreme ends of bars forming a simple U or multiple stirrups shall be anchored by one of the methods of Section 307(h)1 or shall be bent through an angle of at least 90° tightly around a longitudinal reinforcing bar not less in diameter than the stirrup bar, and shall project beyond the bend at least 12 diameters of the stirrup bar.

3. The loops or closed ends of such stirrups shall be anchored by bending around the longitudinal reinforcement through an angle of at least 90°, or by being welded or otherwise rigidly attached thereto.

4. Hooking or bending stirrups or separate web reinforcement bars around the longitudinal reinforcement shall be considered effective only when these bars are perpendicular to the longitudinal reinforcement.

5. Longitudinal bars bent to act as web reinforcement shall, in a region of tension, be continuous with the longitudinal reinforcement. The tensile stress in each bar shall be fully developed in both the upper and the lower half of the beam by adequate anchorage through bond or hooks.

(i) Hooks.

1. The term "hook" or "standard hook" as used herein shall mean either:

(1) A complete semi-circular turn with a radius of bend on the axis of the bar of not less than 3 and not more than 6 bar diameters, plus an extension of at least 4 bar diameters at the free end of the bar, or

(2) A 90° bend having a radius of not less than 4 bar diameters plus an extension of 12 bar diameters.

Hooks having a radius bend of more than 6 bar diameters shall be considered merely as extensions to the bars.

2. In general, hooks shall not be permitted in the tension portion of any beam except at the ends of simple or cantilever beams or at the freely supported ends of continuous or restrained beams.

3. No hook shall be assumed to carry a load which would produce a tensile stress in the bar greater than 7,500 psi.

4. Hooks shall not be considered effective in adding to the compressive resistance of bars.

5. Any mechanical device capable of developing the strength of the bar without damage to the masonry may be used in lieu of a hook. Tests must be presented to show the adequacy of such devices.

308. REINFORCED MASONRY COLUMNS AND WALLS

Notation

A_g = The overall or gross area of a reinforced masonry column; the total area of the masonry incasement of combination columns.

A_s = Effective cross-sectional area of reinforcement in compression in columns.

d = The least lateral dimension of a column.

e = Eccentricity of the resultant load on a column measured from the gravity axes.

f'_m = Compressive strength of masonry at 28 days, unless otherwise specified.

f_s = Nominal allowable stress in vertical column reinforcement.

h = Unsupported length of column or wall.

p_v = Ratio of volume of lateral reinforcement to the volume of the masonry core (out to out of ties).

p_g = Ratio of the effective cross-sectional area of vertical reinforcement to the gross area A_g.

P = Total allowable axial load on a column, h/d 10 or less.

P′ = Total allowable axial load on a column, h/d greater than 10.

t = Overall depth of column section.

(a) Minimum Dimensions of Columns. Reinforced masonry columns shall have a minimum thickness of 12 in. Exception: For minor columns not supporting floor or concentrated roof loads, columns may have a minimum thickness of 8 in.

(b) Unsupported Height of Columns.

1. The maximum unsupported height h of masonry columns shall be not more than 25 times the least lateral dimension of the column.

2. The unsupported height h of masonry columns shall be taken as not less than the clear distance between the floor surface and the under side of the deeper beam framing into the column in each direction at the next higher floor level.

3. For rectangular columns, that combination of vertical and horizontal dimensions shall be used which gives the greatest ratio of h/d.

(c) Permissible Load on Columns.

1. The maximum axial load on columns having the ratio p_v more than .006 shall be computed by the following formula:

$$P = A_g\,[0.20\,f'_m + 0.65\,p_g f_s]$$

2. The maximum axial load on columns having the ratio p_v .006 or less shall be 80 per cent of that computed by the above formula.

3. The maximum load P′ on axially loaded columns, having an unsupported length h greater than ten times the least lateral dimension d, shall not exceed:

$$P' = P\left[1.3 - 0.03\,\frac{h}{d}\right]$$

(d) Vertical Reinforcement.

1. The ratio (p_g) of vertical reinforcement shall be not less than .005 nor more than .04. The number of bars shall be not less than 4, nor the diameter less than ⅝ in.

2. Where lapped splices are used, the amount of lap shall be sufficient to transfer the allowable stress by bond, but in no case shall the length of lapped splice be less than 20 bar diameters.

3. Welded splices shall develop the full strength of the bar.

4. The column reinforcement shall be held firmly in its designed position.

(e) Lateral Reinforcement.

1. Lateral reinforcement shall be ties at least ¼ in. in diameter and shall be spaced apart not over 16 vertical bar diameters, 48 tie diameters, the least dimension of the column, or 16 in.

2. In columns where the ratio p_v of lateral reinforcement is more than .006, the maximum center-to-center spacing of the lateral reinforcement shall be 4 in. Lateral reinforcemment shall be not less than ¼-in. bars, shaped as a circle or rectangle and splices shall be made by welding or lapping 50 diameters. Two concentric ties may be placed in one joint.

(f) Bending Moments in Columns.

1. Bending moments resulting from eccentric loads and conditions of restraint as in the case of rigid frames or other forms of continuous construction shall be considered in the design.

2. Columns shall be designed to resist the axial forces from loads on all floors, plus the maximum bending due to loads on a single adjacent span of the floor under consideration.

3. The resistance to bending at any floor level shall be provided by distributing the moment between the columns immediately above and below the given floor in proportion to their relative stiffness and conditions of restraint.

(g) Combined Stresses. Stresses due to combined axial load and bending shall be determined in accordance with Section 306(e), except that, where e/t is less than ¼, design may be based on the uncracked section.

(h) Reinforced Masonry Walls.

1. The allowable axial stress in reinforced masonry bearing walls with minimum reinforcement as required by Section 308(h)5 shall not exceed 0.20 f'_m. When the reinforcement in bearing walls is designed, placed and anchored in position as for columns, the allowable stresses shall be on the basis of Section 308(c) as for columns.

2. Walls shall be designed for eccentric loads and for any lateral forces, pressures or shears to which they are subjected. The design shall conform to the requirements of Section 308(g).

3. Reinforced masonry bearing walls shall have a nominal thickness of at least 1/25 of the unsupported height or width, whichever is the shorter. Panel and enclosing walls of reinforced masonry shall have a thickness of not less than 1/30 the distance between the supporting or enclosing members.

Limits on height-to-thickness ratios of this section may be waived when written evidence is submitted by a qualified person showing that the walls meet all the other requirements of this code.

4. Reinforced masonry walls shall be securely anchored to adjacent structural members, such as roofs, floors, columns, pilasters, buttresses and intersecting walls.

5. Reinforced masonry walls shall be reinforced with an area of steel not less than 0.002 times the cross-sectional area of the wall, not more than ⅔ of which may be used in either direction. The maximum spacing of vertical reinforcement shall be 36 in.

Horizontal reinforcement shall be placed in the top of footings, at the bottom and top of wall openings, at roof and floor levels and at the top of parapet walls. Only horizontal reinforcement which is continuous in the

wall shall be considered in computing the minimum area of reinforcement. In addition to the minimum reinforcement, there shall be not less than one ½-in. diameter bar around all window and door openings, which shall extend at least 24 in. beyond the corner of the openings.

(i) Partially Reinforced Masonry Walls.

1. Partially reinforced masonry shall be designed as for plain masonry, except that reinforcement may be considered as resisting flexural tensile stresses.

2. The minimum area of reinforcement required in Section 308(h)5 shall not apply to partially reinforced masonry walls. Maximum spacing of vertical reinforcement in exterior partially reinforced masonry walls shall be 6 ft. Reinforcement shall be placed each side of each opening and at each corner of all walls. Horizontal reinforcement shall be placed in the top of footings, at the bottom and top of wall openings, at roof and floor levels, and at the top of parapet walls.

3. The effective width used in computing flexural stresses where reinforcement occurs in partially reinforced masonry shall be not greater than 4 times the wall thickness nor more than 1½ times the unit length in hollow masonry.

CHAPTER 4

DESIGN CRITERIA

401. INTRODUCTION

The purpose of this chapter is to present and discuss the most important factors in the design of reinforced brick masonry structures and building units. Since the principal reason for using reinforced masonry in lieu of unreinforced masonry is to obtain greater resistance to lateral forces induced by wind, earthquake, or bomb blast, these will be emphasized. Of course, it will not be possible to provide herein a complete coverage of the many and complex considerations in the subject of structural design for resistance to lateral forces. This could not be done properly in several volumes. It is hoped, however, that the material presented, together with the examples in Chapters 5 and 6, will prove useful to the reader and provide him with some appreciation of how good structural design, good construction and reinforced brick masonry can be combined to offer resistance to forces of nature and man.

402. DEAD LOADS

The weight of all materials in a building or structure constitutes its "dead load" and can be determined quite simply and with much more accuracy than the live loads or lateral forces. In some cases, fixed equipment or machinery may also be considered as a dead or real load since it will be of a definite weight, attached to the structure and therefore in a fixed location. Many building codes specify the unit weights of materials which are to be assumed in the design. The tabulations of Tables 4-1, 4-2 and 4-3 are presented for use where codes are not in force or do not assign unit weights. There is more or less variation in the weights of materials. Weights given in Tables 4-1, 4-2 and 4-3 are approximate averages, and may be used for the purpose of determining dead loads.

TABLE 4-1

WEIGHTS OF BUILDING MATERIALS (VOLUME)

Description of Material	Weight, pcf
Brick masonry, unreinforced	120
Reinforced brick masonry	125
Structural clay tile masonry walls (average)	60
Structural clay tile partitions (average)	50
Concrete:	
Stone or gravel	150
Slag	130
Cinder	115
Haydite	100
Lightweight	70–100

Concrete masonry units:	
Stone aggregate	90
Lightweight aggregate (average)	58
Stone masonry	160
Mortar	116
Plaster	96
Cement, portland, loose	94
Sand (dry)	100
Lime (hydrated)	40
Cinders	45
Steel	490
Cast iron	450
Wood:	
Fir, dry	32
Fir, wet	44
Yellow pine, dry	42
Oak	48
Insulation:	
Rock wool (loose)	10
Wood fiber (rigid)	15
Corkboard	9
Granulated cork	6

TABLE 4-2

WEIGHTS OF BUILDING MATERIALS (AREAS)

Description of Material	Weight, psf
Studs, plates and bridging:	
2x4 @ 12 in. to 16 in.	2
2x6 @ 12 in. to 16 in.	3
Joists:	
2x8 @ 20 in.	2
2x8 @ 14 in. and 2x10 @ 18 in.	3
2x10 @ 12 in. and 2x12 @ 16 in.	4
2x12 @ 12 in.	5
Sheathing	3
Hardwood finish	4
Floor finish of terrazzo, tile, mastic, linoleum, per in. thickness including base	12
Roofing:	
Wood shingles	3
Asbestos shingles	4
Asphalt shingles	6
Slate, ¼ in.	10
Tar and gravel	6
Ready roofing	1
Copper or tin	1
Corrugated iron	2
Clay tile	10–18
Skylights, metal and wire glass	8

Plaster (gypsum):	
On masonry, 5/8 in.	5
On wood lath	8
On metal lath	10
On suspended metal ceiling	10
Solid, 2 in. with metal studs and metal lath	22
Wood studs, 2x4, wood lath or plasterboard, 5/8 in. plaster, two sides	16
Stucco (cement) 1 in. thick	10
Cinder fill, 2 in. thick	7
Cement floor finish, 1 in. thick	12
Asphalt, 2 in. thick	25

TABLE 4-3

WEIGHTS OF MASONRY WALLS AND PARTITIONS

Description	Pounds per Square Foot Nominal Thickness					
	2 in.	3 in.	4 in.	6 in.	8 in.	12 in.
Brick masonry	..	..	37	..	78	120
Structural clay tile walls ①:						
5-in. unit height	..	..	24	34	42	66②
8-in. unit height	..	..	22	32	38	55
12-in. unit height	..	..	21	30	34	49
Brick and tile combination wall	..	..	..	..	63	83
Structural facing tile (glazed or unglazed):						
5-in. unit height	16½	..	30	41	50	..
8-in. unit height	16	..	27	..	..	..
Glazed brick:						
2¼-in. unit height	17	..	37	..	..	..
Clay tile partitions	15	16	17	24	32	46
Concrete masonry units (8-in. unit height):						
Stone and gravel aggregate	..	..	34	50	58	90
Lightweight aggregate (average)	..	..	22	31	36	58
Gypsum partitions	10③	13③	13	19	..	..

① Scored for plaster application.
② Two units in wall thickness.
③ Solid.

Note: No plaster or stucco finishes included in the above weights.

403. LIVE LOADS

All loads other than the dead loads, except the horizontal forces of wind, earthquake, or blast are termed "live" loads. Such live loads may vary greatly from time to time or may be constant for long periods of time as in a storage warehouse. Most building codes permit certain reductions in live loads for members carrying large tributary areas since the chances of extensive areas or floors all being fully loaded at one time are quite remote.

Live loads associated with various occupancies or uses are prescribed in building codes. These loads include a sufficient allowance to cover the effects of ordinary impact; however, for special occupancies involving unusual impacts, such as those resulting from moving machinery, elevators, craneways, etc., a suitable increase in the assumed live loads should be made.

Most building codes contain the requirement that the maximum floor loads for which the building is designed shall be posted "in a conspicuous place in each space to which they relate" and that these loads shall not be exceeded.

The following is reproduced from the "American Standard Building Code Requirements for Minimum Design Loads in Buildings and Other Structures", developed by American Standards Association's Sectional Committee A58 under the sponsorship of the National Bureau of Standards, and published by the National Bureau of Standards in 1945 as Miscellaneous Publication M179.

"3-1. Uniformly Distributed Floor Loads.

"(a) The live loads assumed for purposes of design shall be the greatest loads that probably will be produced by the intended occupancies or uses, provided that the live loads to be considered as uniformly distributed shall be not less than the following:

Occupancy or Use	Live Load psf
Apartment houses:	
Private apartments	40
Public stairways	100
Assembly halls:	
Fixed seats	60
Movable seats	100
Corridors, upper floors	100
Corridors:	
First floor	100
Other floors, same as occupancy served except as indicated	
Courtrooms	80
Dance halls	100
Dining rooms, public	100
Dwellings	40
Hospitals and Asylums:	
Operating rooms	60
Private rooms	40
Wards	40
Public space	80
Hotels:	
Guest rooms	40
Corridors serving public rooms	100
Public rooms	100
Loft buildings	125
Manufacturing, light	125
Office buildings:	
Offices	80
Lobbies	100

Occupancy or Use	Live Load psf
Schools:	
Classrooms	40
Corridors	100
Stores	125
Theaters:	
Aisles, corridors, and lobbies	100
Orchestra floor	60
Balconies	60
Stage floor	150

"(b) When occupancies or uses not listed above are involved, the live load shall be determined in a manner satisfactory to the building official.

"3-2. *Provision for Partitions.* In office buildings or other buildings where partitions might be subject to erection or rearrangement, provision for partition weight shall be made, whether or not partitions are shown on the plans, unless the specified live load exceeds 80 lb. per sq. ft.

"3-3. *Concentrated Loads.* In the design of floors, consideration shall be given to the effects of known or probable concentrations of load to which they may be subjected. Floors shall be designed to carry the specified distributed loads, or the following minimum concentrations, whichever may produce the greater stresses. The indicated concentrations shall be assumed to occupy areas 2½ ft. sq. and to be so placed as to produce maximum stresses in the affected members.

"Office floors, including corridors	2,000 lb.
Garages	Maximum wheel load
Trucking space within building	Maximum wheel load

"3-4. *Partial Loading.* When the construction is such that the structural elements thereof act together in the nature of an elastic frame due to their continuity and the rigidity of the connections, and the live load exceeds 150 lb. per sq. ft. or twice the dead load, the effect of partial live load such as will produce maximum stress in any member shall be provided for in the design.

"3-5. *Impact Loads.* The live loads specified in Section 3-1(a) may be assumed to include a sufficient allowance to cover the effects of ordinary impact. For special occupancies and loads involving unusual impacts, such as those resulting from moving machinery, elevators, craneways, vehicles, etc., provision shall be made by a suitable increase in the assumed live load.

"3-6. *Reduction of Live Load.*

"(a) No reduction shall be applied to the roof live load.

"(b) For live loads of 100 lb. or less per sq. ft., the design live load on any member supporting 150 sq. ft. or more may be reduced at the rate of 0.08 per cent per sq. ft. of area supported by the member, except that no reduction shall be made for areas to be occupied as places of public assembly. The reduction shall exceed neither R as determined by the following formula nor 60 per cent.

$$R = 100 \times \frac{D+L}{4.33L}$$

in which R = reduction in per cent

D = dead load per square foot of area supported by the member

L = design live load per square foot of area supported by the member.

For live loads exceeding 100 lb. per sq. ft., no reduction shall be made, except that the design live loads on columns may be reduced 20 per cent.

"3-7. *Restrictions on Loading.* It shall be unlawful to place, or cause or permit to be placed, on any floor or roof of a building or other structure a load greater than that for which such floor or roof is designed.

"3-8. *Posting of Live Loads.* In every building or other structure, or part thereof, used for mercantile, business, industrial, or storage purposes, the loads approved by the building official shall be marked on plates of approved design which shall be supplied and securely affixed by the owner of the building, or his duly authorized agent, in a conspicuous place in each space to which they relate. Such plates shall not be removed or defaced but, if lost, removed, or defaced, shall be replaced by the owner or his agent.

"3-9. *Roof Loads (including snow loads).*

"(a) Ordinary roofs, either flat or pitched, shall be designed for a load of not less than 20 lb. per sq. ft. of horizontal projection in addition to the dead load, and in addition to either the wind or earthquake load, whichever produces the greater stresses."

NOTE. The figure of 20 lb. per sq. ft. is a minimum. In preparing local codes, weather bureau data should be consulted and the indicated snow load for the locality, if larger, substituted for this minimum.

"(b) Roofs to be used for promenades shall be designed for a minimum load of 60 lb. per sq. ft. in addition to the dead load. Roofs to be used for other special purposes shall be designed for appropriate loads as directed or approved by the building official.

"3-10. *Other Live Loads.*

"(a) Stair treads shall be designed to support a uniformly distributed load of 100 lb. per sq. ft., or concentrated loads of 300 lb. spaced 3 ft. center to center each occupying an area 1 ft. wide by the depth of the tread, whichever will produce the greater stresses.

"(b) Sidewalks shall be designed to support either a uniformly distributed load of 250 lb. per sq. ft., or a concentrated load of 8000 lb. on an area 2½ ft. sq. placed in any position, whichever will produce the greater stresses.

"(c) Driveways shall be designed to support a uniformly distributed load of 100 lb. per sq. ft. for vehicles weighing less than 3 tons with load, 150 lb. per sq. ft. for vehicles weighing 3 to 10 tons with load, 200 lb. per sq. ft. for vehicles weighing over 10 tons with load, or a concentrated load equal to the maximum expected wheel load on an area 2½ ft. sq. placed in any position, whichever will produce the greater stresses.

"(d) Accessible ceilings, scuttles, and ribs of skylights shall be designed to support a concentrated load of 200 lb. occupying an area 2½ ft. sq., and so placed as to produce maximum stresses in the affected members.

"(e) Stairway and balcony railings, both exterior and interior, shall be designed to resist a horizontal thrust of 50 lb. per lin. ft. applied at the top of the railing.

"3-11. *Load Tests.* The building official may require a load test of any construction whenever there is reason to question its safety for the intended occupancy or use."

In contrast with the roof live loads required in the above, the 1952 edition of the Uniform Building Code contains the following live load requirements for roofs:

"Section 2305. Roofs shall sustain, within the stress limitations of this Code, all 'dead loads' plus unit 'live loads' set forth in Table 4-4. The live loads shall be assumed to act vertically upon the area projected upon a horizontal plane.

"Greenhouses, lath houses and agricultural buildings shall be designed for a vertical live load of not less than 10 lb. per sq. ft.

TABLE 4-4

ROOF LIVE LOADS IN POUNDS PER SQUARE FOOT

Tributary Loaded Area in Square Feet For Any Structural Member	0 to 200	201 to 600	Over 600
Roof Member			
Flat or rise less than 4 in. per ft. Arch or dome with rise less than 1/8 of the span	20	16	12
Rise 4 in. per ft. to less than 12 in. per ft. Arch or dome with rise 1/8 span to less than 3/8 span or with radius 3/4 or greater of the span	16	14	12
Rise 12 in. per ft. and greater. Arch or dome with rise 3/8 span or greater, or radius less than 3/4 the span	12	12	12

"Trusses and arches shall be designed to resist the stresses caused by unit live loads on one-half of the span if such loading results in reverse stresses, or stresses greater in any portion than the stresses produced by the required unit live load upon the entire span. For roofs whose structure is composed of a stressed shell, framed or solid, wherein stresses caused by any point loading are distributed throughout the area of the shell, the requirements for unbalanced unit live load design may be reduced 50 per cent.

"When the form factor, as determined by wind tunnel tests or other recognized methods, indicates vertical or horizontal loads of lesser or greater severity than those produced by the loads herein specified, the roof structure may be designed accordingly.

"Snow load, full or unbalanced, or wind load shall be considered in place of loads in Table 4-4, where such loading will result in larger members or connections."

The matter of snow load and roof slopes should be given particular attention, especially at locations where snow is common. The actual loading may be considerably greater than roof live load code requirements.

Live load requirements of the 1952 Uniform Building Code are given in Table 4-5.

TABLE 4-5

UNIT LIVE LOADS

Occupancy	psf
Apartments	40
Armories	150
Auditoriums—Fixed seats	50
Movable seats	100
Balconies and Galleries—Fixed seats	50
Movable seats	100
Cornices	60
Corridors, Public	100
Dance Halls	100
Drill Rooms	100
Dwellings	40
Exterior Balconies	100

Fire Escapes	100
Garages	100
Gymnasiums	100
Hospitals—Wards and rooms	40
Hotels—Guest rooms and private corridors	40
Libraries—Reading rooms	60
Stack rooms	125
Loft Buildings	100
Manufacturing—Light	75
Heavy	125
Marquees	60
Offices	50
Printing Plants—Press rooms	150
Composing and linotype rooms	100
Public Rooms	100
Rest Rooms	50
Reviewing Stands and Bleachers	100
Roof Loads (See Table 4-4)	
Schools—Class Rooms	40
Sidewalks	250
Skating Rinks	100
Stairways	100
Storage—Light	125
Heavy (Load to be determined from proposed use or occupancy, but never less than)	250
Stores—Retail (Light merchandise)	75
Wholesale (Light merchandise)	100

"All ceiling joists shall be designed for not less than 10 lb. per sq. ft. total load.

"All balcony railings shall be designed to withstand a horizontal force of 20 lb. per lin. ft., applied at the top of the railing."

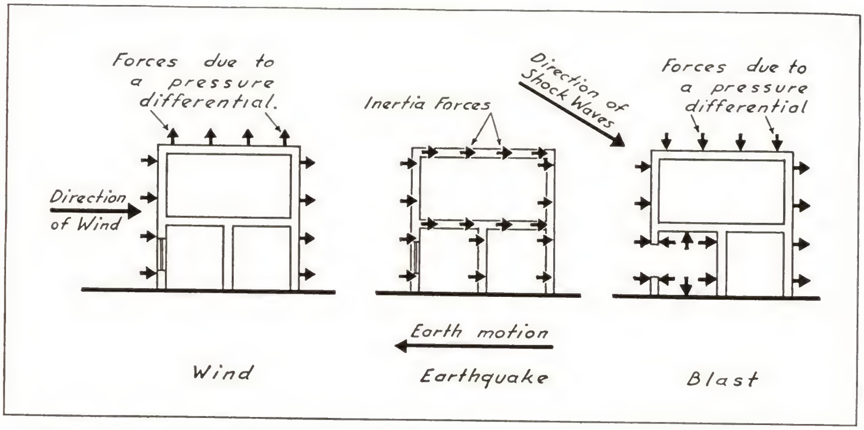
404. LATERAL FORCES—GENERAL CONSIDERATIONS

(a) Comparison of Lateral Forces. The common characteristic of forces induced by wind, earthquake and blast is that they are dynamic in character; i.e., they are caused by motion of either the air as a fluid or the ground in case of earthquakes and perhaps blasts. The time factor is most important, particularly for the earthquake and bomb blast. Although maximum velocity of the wind occurs in gusts of very short duration, generally speaking, wind loading is more sustained than either the earthquake or the blast loading.

The wind may strike a building from any direction and not necessarily normal to the wall surfaces. In many cases, particularly in cities where there are many adjacent structures, the wind may approach a particular wall or surface of a building from almost any angle. Blast forces are like wind in that they have a common source or point from which the disturbance originates. However, the time duration of blast action is much shorter; in fact, the maximum loading occurs in a matter of hundredths of a second. A bomb may explode at the ground surface or at any elevation above the ground, and in some cases may explode below the ground. Where the blast occurs in, on, or close to the ground, considerable ground motion may be transmitted to the structure as well as air disturbance. This fact, plus the fact that the blast is of relatively short duration (has an impact effect) and also reverses direction, gives the earthquake and blast reaction something in common. Blast disturbances of the air, as well as windstorm disturbances, are aerodynamic in nature and involve many considerations such as shape factor, suction effect, drag, etc.

The earthquake results from a sudden release of energy which causes motion in the ground surface. The necessary potential energy is developed by geological movements or strain in the earth's crust. The most common release of this energy is by sudden movement along fault lines or cracks. These movements may be horizontal or vertical, or a combination. The sudden slipping along a fault line propagates waves through the earth from the point of slip, or the epicenter. Because of these waves being reflected from various points in the earth and because of the various types of soil or rock through which the various waves pass, or rebound, the motion as it strikes a structure is usually very chaotic and irregular and contains not only both lateral components but the vertical as well. The inertia of the structure and its various parts causes forces to develop. Generally speaking, the more resistance in mass and rigidity, the more forces induced. If the force cannot be sustained, damage or even complete collapse will occur.

Fig. 4-1 is an over-simplified illustration of the forces on a hypothetical structure from wind, earthquake and blast. No scale or quantitative comparison is intended. The intensities of the various forces will vary or possibly reverse with time and according to various conditions.



Note. The direction of shock waves and the conditions indicated for blast are for the case where the structure is no farther from ground zero than height of the bomb burst. If this is not the case, the blast wave will probably travel horizontally.

Fig. 4-1

Types of Lateral Forces

(b) The Assumption of Static Lateral Forces. Because of the very complex nature of dynamic forces, it has become customary in wind and earthquake-resistant design to substitute assumed static lateral forces or pressures for the true transitory dynamic forces and pressures. In our present state of knowledge in designing against blast forces, the same approach is usually taken. This method is practical and is recommended at this time for ordinary structures, providing: (1) the equivalent static forces have been properly determined; (2) they are properly applied in design; and (3) in the design, there is realization of the true nature of the forces so that important details, stress paths, connections, etc., can be provided.

Special structures, such as tall slender buildings, stacks, towers, bridges and others, require much more than the sole use of equivalent static lateral forces for successful design and economic construction.

Much progress has been and is being made by engineers to improve the general knowledge and methods of design for earthquake-resistant structures. These improvements consist, in part, of changing the amount of the assumed lateral forces, not only for different buildings and localities, but often for different heights or stories of a tall, slender structure. The attempt is to make the assumed static lateral forces more realistic of the envelope or maximum effects of the actual dynamic forces and still keep design methods practical. There are, of course, many more considerations in design for lateral forces than the determination of the lateral force to be assumed, whether the design be to resist wind, earthquake or blast. Some of these considerations will be discussed subsequently.

(c) Combination of Lateral Forces. It is never required or necessary that more than one of the three mentioned lateral forces be used in design combinations. The chances of any two of violent wind storm, severe earthquake, or a bomb blast occurring at any one instant are so remote as to be neglected as a calculated risk. Nor is it necessary to apply the forces in more than one direction at a time. Other lateral forces may be produced in certain structures by moving equipment, such as traveling cranes, machinery or vehicles. These are special considerations and their full amount being combined with maximum wind, earthquake or blast forces would be a very conservative assumption, since during an earthquake, bomb blast or a violent wind storm practically all normal activities would cease.

405. WIND FORCES

The most common cause of lateral force on buildings is wind, the maximum velocity of which varies greatly from place to place. In most cases unreinforced masonry construction has shown adequate resistance to wind. There are locations, however, where very great wind velocities must be resisted, or where relatively lightweight masonry construction is used, which necessitate special consideration and the use of modern reinforced brick masonry.

The history of damage to structures by wind, hurricane and tornado is very extensive. Building codes all over the world set forth minimum standards of design intended to make structures safe against damage or collapse from the wind forces to be expected in each locality. In spite of these requirements and the extensive history of damage, each new year produces cases of wind damage or destruction. The reasons for this are many, including (1) unprecedented wind velocities or pressure drops strike certain areas, (2) the structures become weakened from age, disrepair or rot in their resistance to normal maximum wind forces, (3) faulty design and/or construction, or (4) buildings are erected beyond the limits of jurisdiction of the building regulations without proper design and construction. In many cases the building regulations may be inadequate or not intended to cope with unusual structures or structures with an unusual exposure to wind, or the conventional static pressure design method fails to provide for the true aerodynamic phenomenon.

The general use of formulae which produce static lateral pressures as a function of a constant times the square of the velocity of the wind, or the code requirements of providing for so many pounds per square foot of wind pressure without even mentioning wind velocity or exposure, has caused designers in a great many cases to lose sight of the fact that wind pressures are not static but dynamic. Fortunately, this oversight causes no trouble in most cases, particularly for heavier construction such as masonry and concrete buildings. However, special structures, such as towers, poles, stacks or even

buildings, that have an unusual "fetch" or exposure to the wind may be subject to damage or failure if designed under the normal static pressure assumption without consideration of the real aerodynamic problem.

A great deal more has been learned within the last two or three decades about the reaction of wind on structures. Experiments have been made in wind tunnels and also by measuring pressure at various points on actual structures during storms. It has been found that pressure depends upon the size and shape of the structure in not only the face exposed to the wind but in its third dimension. Moreover, there is often a considerable suction (negative pressure) on roof surfaces and on leeward walls, the magnitude of which is dependent upon the shape and size of the building, its roof pitch, adjacent obstructions, the direction of wind and other factors. Torsional effects caused by wind phenomena can be considerable when the wind direction is oblique to the walls or the vertical resisting elements are asymmetrical.

It is quite possible to design and construct structures to withstand wind of even tornado or cyclone intensity, but this requires careful attention to the laws of aerodynamics and details of design, as well as a realistic determination of the maximum wind velocity V, in miles per hour for each particular location, and the suction effects to be expected.

Formulae for determining equivalent static pressure resulting from wind are of the general form:

$$P = KV^2$$

where P is the equivalent static pressure in pounds per square foot, K a constant and V the design wind velocity in miles per hour.

The equivalent static pressure or design wind pressure is a function of the kinetic energy of the moving air and is affected principally by the shape of the structure. Kinetic energy of the moving air, frequently referred to as velocity pressure, is obtained from the fundamental formula of mechanics:

$$K.E. = \tfrac{1}{2} MV^2$$

where K.E. = kinetic energy
M = the mass of the moving object
V = the velocity.

Assuming the weight of air as 0.07635 lb. per cu. ft., and substituting in the above formula with consistent units, the kinetic energy or velocity pressure, q_o, is given by the formula:

$$q_o = 0.00256\ V^2$$

where q_o is pressure in pounds per square foot and V is design wind velocity in miles per hour.

The design wind velocity should be based on weather bureau reports of recorded wind velocities in the area in question. The maximum recorded velocity for which a structure should be designed will depend upon the frequency with which this velocity occurs, the occupancy of the structure, the cost of construction, and perhaps other factors upon which the designer must base a calculated risk.

Wind velocities reported by the weather bureau are usually 5-min. average velocities indicated by a 4-cup anemometer. However, since wind velocity is not constant but is characterized by gusts of varying velocity and of short duration, a so-called gust factor must be applied to average wind velocities to obtain maximum velocity.

Gust factors decrease with height; however, for heights corresponding to those of typical buildings, the decrease is slight. Factors recommended in

the appendix of the American Standard Building Code Requirements for Minimum Design Loads in Buildings and Other Structures are 1.5 at 30-ft. height, 1.3 at 500-ft. height, and 1.23 at 1000-ft height.

Since wind velocities increase with height above the ground, a further correction of weather bureau velocities is necessary in order to determine maximum velocity at any specified height. Data reviewed by W. W. Pagon in the May 23, 1935 issue of Engineering News Record, indicate that the variation of wind velocity is approximately as the 7th root of the height and may be obtained from the formula:

$$V_h = V_w \times \left(\frac{h}{w}\right)^{1/7}$$

where V_h = wind velocity at height h

V_w = wind velocity reported by weather bureau

h = height above ground of point under consideration

w = height above ground of weather station anemometer.

As an example of the use of the equation, assume that the weather bureau anemometer, having a height above ground of 30 ft., records an average wind velocity of 60 mph over 5-min. duration and that it is desired to obtain the average wind velocity at a height of 120 ft. Substituting in the above formula:

$$V = 60 \times \left(\frac{120}{30}\right)^{1/7} = 73.$$

This velocity of 73 mph is the average velocity at 120 ft. and, to obtain the design velocity, a gust factor of 1.5 should be applied, giving a design velocity of 110 mph.

After determining design velocity, velocity pressure may then be obtained from the formula previously stated: $q_o = 0.00256\ V^2$

The equivalent static pressure or the design pressure is then obtained from the velocity pressure by applying a shape factor. This factor depends upon the shape of the structure and is determined largely from wind tunnel tests. For most buildings the value will lie beween 1.3 and 1.5.

Applying the value of 1.3 to the equation for velocity pressure, we have the familiar equation for design pressure:

$$P = 0.00333\ V^2$$

where P is design pressure in pounds per square foot and V is wind velocity in miles per hour.

Table 4-6 reproduced from American Standard Minimum Design Loads in Buildings and Other Structures, A58.1-1945, gives recommended minimum design wind loads for buildings of various heights. These pressures are based on a design wind velocity of 75 mph, corresponding roughly to a 5-min. average of 50 mph at 30 ft. above ground. In the absence of data from which to compute design wind pressures as previously outlined, it is recommended that these values be considered minimum values.

The Uniform Building Code, 1952 edition, contains the following wind force requirements:

"*Wind Pressure.* For purposes of design the wind pressure shall be taken upon the gross area of the vertical projection of buildings and structures at not less than 15 lb. per sq. ft. for those portions of the building less than 60 ft. above ground and at not less than 20 lb. per sq. ft. for those portions more than 60 ft. above ground.

"The wind pressure upon roof tanks, roof signs, or other exposed roof structures and their supports shall be taken as not less than 30 lb. per sq. ft. of the gross area of the plane surface, acting in any direction. In calculating the wind pressure on circular tanks, towers or stacks, this pressure shall be assumed to act on six-tenths of the projected area.

"On open framed structures, the area used in computing wind pressure shall be 1½ times the net area of the framing members in the side exposed to the wind.

"Greenhouses, lath houses and agricultural buildings shall be designed for a wind pressure of not less than 10 lb. per sq. ft."

TABLE 4-6

DESIGN WIND PRESSURES FOR VARIOUS HEIGHT ZONES OF BUILDINGS OR OTHER STRUCTURES

Height Zone ft.	Wind Pressure psf	Height Zone ft.	Wind Pressure psf
Less than 50	20	600 to 799	35
50 to 99	24	800 to 999	36
100 to 199	28	1000 to 1199	37
200 to 299	30	1200 to 1399	38
300 to 399	32	1400 to 1599	39
400 to 499	33	1600 and over	40
500 to 599	34		

406. FORCES INDUCED BY EARTHQUAKES

Earthquake forces are induced by ground motion. The inertia effect of the mass of the structure, i.e., the tendency of a body at rest to remain at rest, results in stresses. Generally speaking and subject to dynamic phenomena, the greater the weight and rigidity of the structure, the greater are the forces which must be resisted by the structure before failure occurs.

As discussed in Section 404, earthquake motion is violent and chaotic, but for practical reasons design for earthquake resistance is generally done with assumed static lateral forces. Much has been written about treating the earthquake problem in a dynamic sense by assuming a simple one (or few) mass system under sinusoidal motion. The trouble with this method is that the motion of the ground is usually far from sinusoidal because of its chaotic nature. To reduce it to such motion would involve perhaps hundreds of superimposed sinusoidal waves of different periods and amplitudes. Another factor is that a building has many degrees of freedom for each story and some buildings have a great many stories. Moreover, the natural periods of vibration of buildings are inclined to change under earthquake motion, due to yielding of certain parts or the bringing into play of more parts as the motion continues. The phenomenon of resonance can and does occur to a limited degree in earthquakes, but fortunately its extent in buildings is apparently greatly curtailed by damping of the vibrating structure and by changes in period of the ground and, to some extent, of the structure.

Considerable research has been and is being done on this complex and interesting problem by many individuals, organizations and agencies too numerous to mention here. There is no question that our knowledge of earthquakes, the motion resulting from earth movements, and of the vibrational phenomena involved, will continue to advance. Code and design method changes

can be expected from time to time. Earthquake-resistant design cannot be entirely reduced to code requirements or forces or mechanical processes in this day and perhaps for a great many years to come. It is essentially an art which takes either the code requirements for assumed static lateral forces or the designer's forces (should there be no applicable codes or should the designer have in mind forces greater than the minimum code requirements for special cases) and treats them not only mathematically but with considerable judgment which takes into account the true nature of the earthquake motion and forces and the history of damage.

The customary present-day approach to seismic design is to apply horizontal force, in each direction alternately, and to each principal mass, such as a story of a building, as follows:

$$F = CW$$

where $F =$ the lateral force, C is a coefficient or seismic factor, and W is the weight of the unit under consideration.

The coefficient C varies to some extent between various building codes and also, in some cases, with the geographical location, the type of soil, and the number of stories of a building. The various factors, or coefficients C, have been established from time to time by engineers specializing in the field, building officials, and seismologists who have considered the damage to structures and also structures which have gone through severe earthquakes with little or no damage. They have also taken into account the records of actual earthquake motion which have been obtained in recent years; the dynamic behavior of buildings as shown by laboratory, mathematical and field research; the frequency of disastrous earthquakes; and the particular hazard involved for each type of structure. In addition to these considerations, the additional cost of providing protection must be a consideration since providing costly protection for a disaster that may occur very infrequently, or perhaps never during the life of the structure, may not be justified.

It is not expected that the use of earthquake design coefficients and methods necessarily provide complete freedom from all damage in the event of a major earthquake, but it is expected, and this has been shown to be the case in all recent earthquakes, that buildings properly designed and constructed to resist earthquakes do so with little or no damage and, in most cases, are constructed at little or no additional cost.

The perimeter of the Pacific Ocean is generally considered the most active earthquake area, although very severe shocks have occurred elsewhere including Italy and India. Contrary to popular belief, all seismic activity in the United States has not been limited to California and the Pacific Coast.

The earthquake requirements of a number of building codes that have been developed during recent years are given in Appendix A and the reader will note a considerable variation in the amount of coefficient C in the various code requirements.

There are many reasons for this, including the fact that the amount of earthquake resistance designed and built into the structure involves many other considerations than the coefficient C alone. These include the amount of live load which is added to the dead load when determining the weight W; the allowable stress increase permitted when considering lateral forces (all codes quoted allow ⅓ increase in stress, although other proposed codes in California and also active codes in Japan have been based upon much higher stress increases). Other questions are whether or not certain structures must

have a frame with built-in resistance or whether the resistance can be provided entirely by the walls, floors and roof in "box action". Another consideration is the scope or intent of the code, whether it is to protect life and major property damage from catastrophes of infrequent occurrence, or whether it is to provide structures that will "come through" a very severe earthquake without even minor non-structural damage. Obviously public buildings involve more responsibility than private structures.

These fundamental considerations, together with others and the fact that various engineers and officials approach these problems from different viewpoints and knowledge of the nature of earthquakes and their effects on structures, account for differences of opinion. More data are still to be obtained from future destructive shocks. Those who have devoted most of their time in study, research, design and observation in the field of engineering seismology feel they have much more to do. Current codes should be considered as representative of the available knowledge and opinion at this time and subject to change.

Some engineers in the San Francisco Bay area, realizing various discrepancies between the resistance of buildings which have survived earthquakes and the requirements of building codes, and also the great variation between codes, undertook to review the entire problem with a fresh outlook and in light of all currently available knowledge. A ten-man committee, representing jointly the San Francisco Section of the American Society of Civil Engineers and the Structural Engineers Association of Northern California, worked diligently on this problem for over two years.

The committee prepared a report which included a suggested lateral force code. This material was published in ASCE Transactions, Vol. 117, 1952. The method of determining the static lateral forces, as proposed in this report and model code, takes into account not only the height of a structure but to some extent its dynamic properties. Moreover, it was based qualitatively at least on actual earthquake records. This code is also unique in that the forces obtained are applied to a structure in a different manner than by other codes. This procedure is also based upon dynamic considerations.

Here again the direct comparison of the coefficient C becomes even less significant, since the moments and shears in a structure will vary in the height depending not only on the forces but their point of application. The following requirements for the determination and application of the lateral forces on buildings are taken from the proposed code prepared by this committee and are recommended herein as the most modern earthquake design requirements (minimum) for buildings in seismic areas.

"*Minimum Earthquake Forces for Buildings* (Section 03)

"(a) *Total Lateral Force.* Every building shall be designed and constructed to withstand minimum total lateral forces, determined independently in the directions of the principal axes of the building, as given by the formula

$$V = CW$$

in which V is the total lateral force or shear at the base; C is the numerical coefficient as given hereafter; and W is the total weight of the building above the base, including dead load plus a percentage of the live load hereinafter specified.

"*(b) Coefficient C.*

"1. In the building as a whole the coefficient C shall be

$$C = \frac{0.015}{T}$$

in which T is the fundamental period of vibration of the building in seconds in the direction considered. The required value of C shall not be less than 0.02 nor more than 0.06.

"2. Qualified persons may submit technical data to substantiate the period T obtaining for a contemplated building. In the absence of such data, it may be assumed that

$$T = 0.05 \frac{H}{\sqrt{b}}$$

in which H is the height of the main portion of the building, in feet, measured above the base which shall be the level at which the structure is positively connected to the ground; and b is the width, in feet, of the main portion of the building in the direction considered.

"(c) *The Weight W.* The weight W of buildings shall include all dead load plus 50 per cent of the design live load for storage and warehouse floors, 25 per cent of the design live load for all other floors, and no live load for roofs.

"(d) *Applied Lateral Forces.*

"1. The total lateral force V shall be distributed over the height of the building in accordance with the following formula:

$$F_x = V \frac{w_x h_x}{\Sigma(wh)}$$

in which F_x is the lateral force applied to any level x; w_x is the weight at level x; h_x is the height of level x above the base; and $\Sigma(wh)$ is the summation of the products wh for the building.

"2. At each level x, the force F_x shall be applied over the area of the building, in accordance with the mass distribution on that level.

TABLE 4-7

COEFFICIENTS C_p

Part	C_p	Comment	Direction
Exterior walls and other non-resisting elements not otherwise classified	0.20	With a minimum of 15 lb. per sq. ft.	Normal to surface of wall
Interior walls and partitions..	0.10	With a minimum of 5 lb. per sq. ft.	Normal to surface of wall
Parapet walls, exterior and interior ornamentations	0.50	....	Any direction horizontally
Towers and tanks, including contents, chimneys, smoke-stacks, and penthouses when connected to or part of a building	0.20	....	Any direction horizontally

"(e) *Lateral Forces on Parts of the Building.*

"1. Parts of buildings and their anchorage shall be designed for lateral forces determined from the formula

$$F_p = C_p W_p$$

in which F_p is the lateral force on the part and in the direction under consideration; W_p is the weight of the part; and C_p is a coefficient selected from Table 4-7 (Page 139).

"2. The distribution of these forces shall be according to the gravity loads pertaining thereto."

The authors recommend a minimum value C of 0.08 for one-, two- and three-story wall-bearing buildings without a complete frame, in lieu of Section 03(b) above.

407. BLAST FORCES

Although a certain amount of information on blast forces can be developed by calculations involving the sudden release of energy, the principal source of design knowledge is empirical. Controlled tests, as well as experience gained on actual structures during war-time, constitute the basis of our present knowledge. At this time (1953) some test data on the effects of atomic bombs have been released. It is hoped and expected that in the near future the responsible agencies will be able to provide the building industry with more information on damage caused by atomic bomb blasts and means of preventing same at various distances from ground zero, the point on the earth immediately below the detonation.

Bombs vary greatly in their effects on structures, according to their type and the location of the point of detonation with relation to the structure under consideration. Some bombs are designed for maximum penetration before exploding, while others have little or no penetrating ability except that from the released blast energy. Other bombs are basically incendiary, while other so-called "fragmentation" bombs are designed to produce maximum injury to personnel and the effects on structures are but nominal.

It has long been recognized that the effects of direct hits of even medium size pre-atomic bombs are so devastating that the cost in materials and money of providing protection against same is very great. With modern aerial mines and atomic bombs it becomes practical in most cases only to design for explosions at a certain distance from the structure under consideration and to consider closer "misses" or direct hits as a risk of warfare.

An example of the nature of forces involved is the explosion of an ordinary (non-atomic) 500-lb. bomb 50 ft. from a structure. This explosion creates a force of approximately 1000 lb. per sq. ft. as compared to the normal 15 or 20-lb. per sq. ft. design for wind. This disparity is not quite as great as it seems for reasons that will be discussed later. The suction effect immediately following the positive pressure from the same bomb will probably reach 1/5th of the maximum positive pressure or about 200 lb. per sq. ft. This explains why, in many cases, where a wall or member cannot be pushed in, it can be pulled out by the suction wave following the initial blast wave. Because of the aerodynamic phenomena involved, it is unlikely that the average pressure over a structure would amount to more than a fraction of the above forces.

In addition to providing against severe damage or failure of a structure or part of a structure from blast effects, consideration should be given to providing reasonable resistance to penetration of fragments, as well as resistance

to fire and nuclear radiation. A great deal of information on the subject matter of air-raid shelters and the effects of bomb blasts on structures has been collected by our military forces and is available to qualified people. The Federal Civil Defense Administration is preparing and releasing the unclassified data on building against atomic attack.

Although there is a tremendous amount yet to be learned about the effect on structures of bombs, particularly atomic bombs, and the problem is quite involved, the design for blast resistance has at least two advantages over the problem of designing for earthquakes. First, a great deal that has been learned about earthquake-resistant design and construction can be applied with judgment to the blast problem; and second, test explosions can be repeated at frequent intervals and under control to increase the field of knowledge. Although much has been learned in the earthquake field by means of shaking table tests, forced vibration of structures, mathematics and other means, the occurrence of destructive earthquakes is so seldom, and then at times and places unknown in advance, that those active in the field of engineering seismology must be patient in waiting for field data and confirmation of design criteria.

Reinforced brick masonry construction is ideal in providing economical resistance to blast forces, penetration of projectiles or flying fragments and nuclear radiation. Reinforced brick walls properly tied to the floors and roof of a building not only provide a great deal of resistance to positive blast effects but also to negative suction effects resulting from blasts. All other things being equal, a building of reinforced brick masonry would be expected to resist bomb explosions occurring much closer than an unreinforced masonry structure where little or no thought is given in design or construction to lateral forces or to tying the walls to the frame, floors and roof. Thus, although absolute protection against bomb blast is practically impossible, at least from the economic standpoint, the proper and more extensive use of reinforced brick masonry should greatly decrease the amount of damage and the number of casualties in a region subject to bomb attack. Any comparison for resistance to lateral forces between the conventional unreinforced unit construction and modern designed and reinforced masonry is meaningless.

Various recommendations have been made by government agencies on assumed static lateral pressures to be used as a basis for design against bombs. Here again the matter is one of calculated risk and how much money can be spent to improve the odds of the structure and its occupants surviving explosive forces.

A booklet, entitled "Damage from Atomic Explosion and Design of Protective Structures", prepared for the National Security Resources Board by the Department of Defense and the U. S. Atomic Energy Commission, was published in 1950. It recommends that the designer assume an equivalent static load of 90 lb. per sq. ft. on vertical surfaces and a downward load of 70 lb. per sq. ft., both on the structure as a whole, in order to provide protection against structural collapse from an atomic bomb releasing energy equivalent to 20,000 tons of TNT exploding at a horizontal distance of ½ mile and a height of approximately 2,000 ft. It is also suggested "that buildings and their component parts be designed employing the methods, allowable stresses, and details employed in wind or earthquake-resistant design". The reader is referred to other parts of this chapter and the following chapters for additional information on lateral force design considerations.

The pressures recommended are as stated for a specific size bomb exploding at a specific point in relation to the building. The inference is that design for greater forces becomes impractical for normal structures (although it could be provided for items of special military or civic value). The document, issued by the National Securities Resources Board, also states: "For greater distances, it is recommended that the designer reduce the pressure in proportion to reduction in peak pressure allowing for the change in vertical angle."

It is to be noted that the peak pressure given for the ½-mile radius is approximately 2,700 lb. per sq. ft., which is a great deal more than the 90 lb. per sq. ft. recommended for design. The apparent discrepancy can be explained by the fact that the peak pressure may not act over the entire exposed surface in one instant of time; that its duration is extremely limited; and that the structure, because of the safety factors of materials, the inertial effect of the structure, elastic and plastic yielding, etc., has a great deal more resistance to shock loading than is implied by conventional design methods and unit stresses. It is also to be realized that a great deal of the recommended material is empirical and is based largely upon the results of atomic bombing in Japan.

The document also presents data as follows regarding the type of damage which might be expected at various distances from ground zero:

Virtually complete destruction	Out to a radius of about ½ mile
Severe to destructive damage	Out to a radial distance of slightly in excess of 1 mile
Moderate to destructive damage	Out to a radius of about 1⅝ miles
Partial damage	Out to a radius of approximately 2 miles
Light damage (mostly plaster damage and window breakage)	Out to a radius of 8 miles or more

Another interesting point is that larger bombs would cause greater damage but not in direct proportion to their equivalent TNT tonnage; for example, a 40,000-ton TNT energy equivalent (100 per cent larger) would increase the above radii by about 25 per cent and the area of damage by about 60 per cent.

The peak pressures given in pounds per square inch at radii corresponding to the above tabulation are approximately as follows:

Distance in miles from ground zero	Peak Pressure
½ mile	19 psi
1 mile (approx.)	6 psi
1⅝ miles	2.7 psi
2 miles	1.9 psi

Although the above are subject to change as additional information is obtained and released regarding the effects of atomic blasts on structures, it is to be noted that the pressures involved are considerable.

Other considerations with blast loading include the fact that not only will the structure as a whole be subjected to the main force of the initial blast wave but a lesser drag force follows which, in turn, is replaced by a comparatively steady force of reverse direction; i.e., toward the point of detonation. This latter is called the suction phase. Moreover, any part of the structure, for example a floor, may be subject to pressure loads of great intensity due to the pressure wave entering one story (through failure of windows or walls) and not entering an adjoining story or basement area. Obviously, many loading situations can develop.

The Federal Civil Defense Administration, in its "Interim Guide for the Design of Buildings Exposed to Atomic Blast", published in June 1952, recommends, in addition to the 90 lb. per sq. ft. horizontal force and 70 lb. per sq. ft. vertical force on the structure as a whole, the following design live loads, all intended to provide "a reasonable chance for survival (of structures) at a distance of ½ mile from ground zero."

Roof and ground floor (slabs and beams)	100 lb. per sq. ft. vertically downward, 70 lb. per sq. ft. upward
Other floors (except shelter area).......	70 lb. per sq. ft. from either side
Walls, interior and exterior (except exterior of shelter areas and those walls expected to fail)	150 lb. per sq. ft. from either side
Slab covering stairwell or elevator shaft	150 lb. per sq. ft. downward.

There are two basic concepts developing in regard to design for resistance to atomic bomb explosion. The first involves an effort to retain the structure as a unit without severe damage to itself or to its contents. The second involves the use of "frangible" walls which will be reduced to small bits and thus not only absorb some energy but greatly reduce the amount of lateral force the remaining structural framework has to resist. Obviously, such design implies that the materials and occupants of the building are either expendable or will be protected in underground or other special smaller structures built within or within close proximity to the major structure.

Whether it is desired to save the entire structure or merely the main structural framework, there is no question that rigidity, redundancy and ductility are extremely important factors. Although the terms "rigidity" and "ductility" may be considered antonyms, the use is slightly different than customary in materials engineering in that the term "rigidity" here implies the providing of transverse shear walls, lateral trusses, etc., which will have not only greater strength but greater rigidity than conventional construction.

"Ductility" refers to the quality of building materials to yield gradually under stress and to undergo considerable deformation before failing. The ability of ductile materials, such as mild steel, to absorb energy above the elastic limit is considerable. Thus, reinforced brick masonry can be expected to stand a much greater explosive force without failure or collapse than an unreinforced masonry construction, because the steel reinforcement provides ductility as well as strength, and reduces the "brittleness" of the masonry.

"Redundancy" in this application may be considered as the quality of a structure to resist further damage or collapse when certain members fail, by

bringing into play other structural elements. Indeterminate frame construction has this quality to a large degree. The use of reinforced brick masonry, wherein walls and other members have resistance to bending in either direction as well as continuity and redundancy, greatly improves the chances of the structure withstanding shocks and motion without collapse.

408. DESIGN CONSIDERATIONS

(a) **Dead and Live Load Design.** Every building as a whole, as well as each member or part thereof, must be adequate to support all dead and live loads without exceeding the maximum allowable unit stresses of the various materials. On the other side of the ledger, it is not desirable to provide more materials than necessary (except as noted below) because of the basic requirement of keeping construction costs to a minimum. The answer to this problem is good design.

Good design takes into account all elements of a problem, not just a few. For example, a design which very frequently reduces member sizes, in order to closely follow calculated stress requirements, may save some materials but costs the owner much more in extra labor. Trade practices, and practical construction methods should be important considerations in design. Other factors, in addition to these and the basic ones of carrying all loads without exceeding allowable unit stresses and cost, include consideration of appearance, utility, deflection, creep or sag, temperature variations, possible future growth of the structure, variations in live loads, and the adaptability of the layout and design for vertical (gravity) loads to be economically applied to the resistance of lateral forces.

Although the design for lateral forces of wind, earthquake and/or blast may be an important factor in the decision to reinforce brick masonry, the dead and live load design must not be neglected. Actually, extreme lateral force loading will occur very seldom, if ever, during the life of the building, whereas all the dead and some of the live load will exist all the time as a steady load.

It is not considered necessary to discuss design methods for dead and live loads. The data supplied elsewhere in this book and others, and the reader's familiarity with the principles of engineering mechanics and design in other materials, should suffice for the design of reinforced brick masonry. The same basic assumptions and methods as used in the design of reinforced concrete are used in the design of reinforced brick masonry. Variations in live load from place to place or span to span should, of course, be considered in determining maximum loads, moments and shears.

An important consideration in reinforced masonry design is the size and shape of the masonry unit and the effect of same on the selected overall sizes of columns, pilasters, beams, etc. After obtaining an approximate idea of the member sizes required, it is desirable to make a scale section of the member with the bars indicated before making final calculations, especially with indeterminate framing.

(b) **The Resistance of Buildings to Lateral Forces.** All buildings, whether so designed or not, have some inherent resistance to lateral forces. This has been shown by the survival of some buildings of little or no designed resistance to severe earthquakes, tornadoes and bombing attacks. The fact of such survival does not guarantee future survivals, no more than does a soldier coming from a battle unwounded guarantee him to be bullet-proof. As emphasized earlier in this chapter, the lateral forces resulting from wind,

earthquake and blast are not uniform static forces or pressures, but are of a chaotic and dynamic nature, unpredictable in the extreme. Peculiar phenomena have and will occur in all such disasters. Moreover, it must be realized that many older buildings, because of their basic layout and integrity of construction, may have considerable lateral force resistance, even though not specifically designed for same.

The dependence upon chance lateral resistance or peculiar dynamic phenomena to save a building or its occupants from disaster in a region subject to extreme lateral forces is dangerous and negligent. The only reliable procedure is to have adequate design for the specified risk or risks by competent engineers familiar with the subject and to insure that the construction is carried out according to the design.

An interesting point is the value of a wind design to resist earthquakes or of an earthquake design to resist blast. Generally speaking, a structure designed to resist heavy wind forces would have some resistance to earthquakes (or vice versa) and, perhaps to a limited extent, to blast. The relationship is not direct in any case and should not be relied upon. For example, a building of a narrow rectangular plan shape, designed for say 20 lb. per sq. ft. wind force in both directions, may have considerable earthquake resistance in the direction parallel to the short sides, but probably would have little earthquake resistance in the other direction. The reason is that the wind force for the latter direction would be based on a narrow face area, whereas the seismic shears would be based upon the entire mass of the structure, regardless of the face exposure. Special design of each building for the specified type and amount of lateral force is the only logical course.

The continued construction of buildings, without design for earthquake, hurricane or tornado in regions subject to same, cannot be condoned. Earthquakes on the Pacific Coast in recent decades have taken a remarkably small loss of life, considering the damage done, principally because the time of occurrence was most fortunate. Such good luck cannot continue indefinitely, and, even if it should, the loss of millions of dollars of property and business time is in itself a most serious matter. Structural engineers have not been surprised at the damage or collapse of most buildings because their construction violated the principles of earthquake-resistant design. In most cases, it is obvious that the buildings were erected merely to "hold up" without any lateral force resistance. In many cases the construction was very poor and one wonders how the structures withstood mild wind forces.

Disasters expose poor designs and poor construction. One of the authors was once privileged to inspect in detail about 80 buildings (mostly wood frame) which were damaged by an explosion a mile or two away. Since the main blast wave had gone overhead, the damage was not too severe; in fact, it was a borderline case as to whether there was damage or not. The interesting thing was that, in general, failure occurred only where there had been poor design, poor materials, poor construction or any combination of the three.

Buildings of any uniform and controlled material can be erected to withstand heavy windstorms or severe earthquakes with little or no damage and with little additional cost. The requirements are (1) good design, and (2) good construction according to the design. The cost of providing for tornadoes or ½-mile misses of atomic bombs will usually be more than for conventional construction, but with good basic layout and resourcefulness, it should not be prohibitive.

(c) Lateral Force Design, General. There are certain basic principles which, although developed in large part for earthquake-resistant design, apply as well to design for severe windstorms and bomb blast. The design for the three types or causes of lateral forces will therefore be treated simultaneously with attention directed to exceptions or specific applications.

Buildings are generally of (1) the bearing-wall type, (2) the frame type, and (3) combinations, having an interior frame construction (columns, beams, girders, slabs, etc.), and exterior bearing walls, quite often of brick masonry. With the pure bearing-wall type, which includes most residences, the lateral force is distributed to the walls parallel to the direction of the force, which in turn transmit their share of the total force in shear to either the foundations or lower stories, as the case may be. The origin, distribution and the determination of the "share" of the lateral force to each such shear wall constitute very important factors in the design.

The frame or "skeleton" type building is usually associated with multi-story office buldings wherein a complete structural framework is erected, floors installed, and so-called curtain or panel walls (non-structural) are placed on the exterior portion of the framework to keep the elements out and the occupants, contents and heat in. With this type of construction we find some very interesting considerations, particularly with regard to earthquake resistance, and more recently in regard to blast resistance.

In both cases, the walls add to the "load". The weight of the walls adds to the seismic force, while the area of the wall "picks up" blast and wind force. On the other hand, the walls can be utilized structurally in both loadings to transmit the forces by shear, thus relieving the frame. Making the walls act structurally increases the rigidity of the building and decreases or eliminates the normal action of the frame under lateral forces because of the relative rigidities of the walls and frame. The more rigid building will tend to have greater seismic shears and will also tend to absorb less blast energy before wall failure. We find, therefore, many arguments pro and con on this complex subject.

The conventional construction of steel frame with brick curtain walls (unreinforced) is usually designed for wind forces in the frame only; i.e., the walls are considered non-structural. Many seismic codes not only require reinforced brick masonry curtain walls but establish such high lateral forces that it is almost impossible for most buildings to provide for same except by making the walls function in shear. In some cases this results in frames designed for vertical loads only and walls designed for lateral forces only. It is the opinion of the authors that, for multi-story buildings, some nominal frame value in resisting lateral forces should be provided in addition to meeting code requirements with rigid shear walls. A very severe impact loading may tend to crack rigid reinforced walls of concrete or brick, in which case a frame having some properties of lateral resistance and energy absorption is greatly to be desired.

In any type of building, the use of reinforcement in the masonry, together with ties to the frame (if any), not only increases the lateral strength of the building and its energy absorption capacity but also means that, in case of very severe loading and wall cracking, the masonry will be held together by its reinforcement and will tend to remain in place. Thus, the structural damage will be reduced and debris will not be thrown to the streets or to the building interior.

It must be obvious to the reader, even thus far, that important basic assumptions based upon experience and professional judgment are necessary before design for severe lateral forces enters the mathematical stage. In many cases two or more schemes or plans for "progressive failure" must be considered. This discussion is not intended to confuse or over-complicate the problem but to present at least a few of the factors involved.

Lateral forces (induced by wind, earthquake or blast) may be taken from their points or levels of "origin" in or on the building to the ground in three basic ways:

(1) By "box action" wherein roof and floor diaphragms transmit and distribute the forces to vertical resisting elements such as shear walls;
(2) By "frame action" wherein framing and/or bracing transmit the developed shears to the foundations;
(3) By combination of box and frame actions.

Most residences and bearing-wall buildings are treated as box action designs, whereas long narrow mill buildings with steel columns and trusses transmit the lateral forces by frame action. Probably the combination of the two is the most common. The decision or method in each case must be settled upon its own merits with particular reference to cost and relative rigidities.

(d) Relative Rigidity. It is amazing how much concern has been developed over the term "relative rigidity", particularly with regard to the earthquake problem. A person regularly engaged in structural design should be familiar with relative rigidity by training or intuition whether or not he were concerned with earthquakes or bomb blasts.

If various elements of a structure are connected to another common rigid element and that element is deflected by a force, then the various connecting units also move (at their point of connection to the common element) an equal amount, or failure occurs somewhere in the system. Assuming failure does not occur, the portion of the load carried by any one of the connecting elements is proportional to its rigidity as compared to the sum of the rigidities of all the connecting elements, or in proportion to its relative rigidity.

In Fig. 4-2, the relationship of the rigidity of a pier and its dimensions is developed for a pier with conditions of fixity at both top and bottom, and for the case where fixity occurs at one end of the pier only.

Fixity, or moment restraint, is provided by horizontal elements at the ends of a pier when they have sufficient strength and rigidity to provide such restraint.

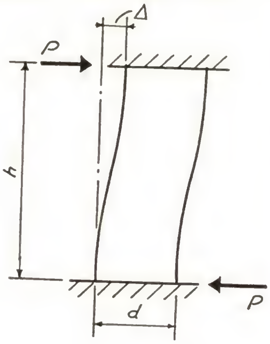
Relative values of the rigidity of piers of varying height to width ratios are plotted in Fig. 5-1 (Page 158) for piers fixed top and bottom and at one end only.

The distribution of a lateral force on a wall to each of the piers is shown in Fig. 4-3.

There are countless examples of relative rigidity considerations, many of which are obvious to the experienced engineer and others which may require tedious calculations of deflection to compare rigidities for a proper assignment of the lateral force to the various structural elements.

The determination of the choice of box vs. frame action, or combinations thereof, may be made by comparison of the rigidity of the horizontal elements (floors and roof) as compared to the vertical elements (frame and walls). A concrete floor system of almost square shape is usually quite rigid as a horizontal beam as compared to the horizontal rigidity of vertical elements.

Pier of thickness, t, fixed top and bottom, showing total deflection, Δ, due to a horizontal force, P. All piers in any one wall having a top connecting element must deflect an equal amount; therefore, the piers will each tend to resist the total applied force in proportion to their rigidities (R_1).



$$\Delta = \Delta_m + \Delta_v$$

Where $\Delta_m = \dfrac{Ph^3}{12E_m I}$ = deflection (inch) due to moment

$\Delta_v = \dfrac{1.2\ Ph}{E_v A}$ = deflection (inch) due to shear

P = horizontal force applied to pier (pound)
h = height of pier (inch)
$I = (1/12)td^3$ = moment of inertia of pier (inch^4)
A = td = area of pier (square inch)
E_m = Modulus of Elasticity of pier (psi)
E_v = Modulus of Rigidity of pier (psi)

Let $E_m = 1 \times 10^6$ and $E_v = 0.4\ E_m$, then

$$\Delta = \frac{Ph^3}{12 \times 1 \times 10^6 \times 1/12 \times td^3} + \frac{1.2\ Ph}{0.4 \times 10^6 \times td} \text{ and,}$$

$$\Delta = \frac{P}{10^6 \times t}\left[\left(\frac{h}{d}\right)^3 + 3\left(\frac{h}{d}\right)\right]$$

Then for a force of P = 1000 lb. on a pier of t = 1 in.,

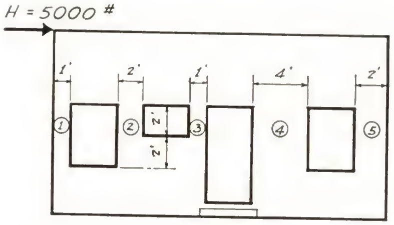
$$\Delta = 0.001\left[\left(\frac{h}{d}\right)^3 + 3\left(\frac{h}{d}\right)\right], \text{ and the rigidity (stiffness)} = R_1 = \frac{1}{\Delta}.$$

For piers that are fixed at one end only,

$$\Delta = 0.001\left[4\left(\frac{h}{d}\right)^3 + 3\left(\frac{h}{d}\right)\right] \text{ since } \Delta_m = \frac{Ph^3}{3E_m I}.$$

Fig. 4-2

Evaluation of Rigidity of a Pier



H = total horizontal force applied to wall.
P = force to each pier and also the design shear.
M = design moment for each pier, where $M = \frac{Ph}{2}$ for top and bottom of piers fixed.

Pier No	h (in.)	d (in.)	h/d	R_1	% of H to ea. pier (R)	P (lb.)	M (in. lb.)
1	48	12	4	13.2	2.0	100	2,400
2	24	24	1	250.	38.2	1910	22,900
3	24	12	2	71.	10.8	540	6,480
4	48	48	1	250.	38.2	1910	45,800
5	48	24	2	71.	10.8	540	13,000
				655.	100.0	5000	

Notes:

a. Each pier is assumed to have full restraint top and bottom and relative rigidity values are taken from the corresponding curve in Fig. 5-1, page 158.
b. Wall returns at each end are assumed to be small and are neglected here.
c. Additional forces to be taken into account in the pier designs are those due to dead and live loads and overturning due to the lateral force ΣP.
d. Note that the lateral force taken by pier No. 2 is the same as for pier No. 4 due to the same value of h/d, but the unit shear stress in pier No. 2 is double that in pier No. 4 due to the relative pier size.
e. Wall assumed to be of constant thickness and, therefore, the value of "t" need not enter into the computations for the determination of "P".

Fig. 4-3
Distribution of Lateral Force to Piers of a Wall

Therefore, the floor qualifies as a diaphragm and may be assumed to carry all horizontal forces to all the vertical members in "box action". If, however, the plan view is long and narrow and if the horizontal element is a wood-sheathed roof, it would be quite flexible and the vertical elements would be expected to take horizontal forces according to their tributary loading, or according to "frame action". Obviously, many intermediate cases occur which involve combinations of the two actions.

(e) Consistency. There is probably no greater virtue in lateral force design than consistency. A scheme or method of dealing with the lateral forces (or perhaps progressive schemes as outlined above) must be established. Once established, the scheme or schemes must be consistently and thoroughly carried out. Every pound of lateral force has to be taken to the ground along some path which is determined by relative rigidities. Each member and each connection between members along each path must be given sufficient capacity to transmit all forces which come to them simultaneously. Usually the connections are the weak points, unless specially designed and indicated on the drawings. The situation is something like a drainage problem where every drop of rainfall on a certain area has to travel according to the greatest slope along its path and eventually reach the low point of the area. Substitute pound for raindrop and rigidity for slope and the crude analogy is obtained.

The use of the word "pound" is not intended to imply extreme accuracy of calculation. Such is not required and would only add to an already

complex job. The important thing is consistency with the basic assumptions and a full realization of the fact that judgment plays a part not only in selecting the stress paths but also the forces themselves. Laborious methods of computation only mislead the designer and others into a false sense of accuracy.

(f) Diaphragms. Horizontal distribution of lateral forces to the various vertical elements at each story may be done through horizontal bracing systems or trusses, by bending of local members from bent to bent, or by utilizing the inherent strength and rigidity of roof and floor constructions. The latter is the diaphragm method and is fundamental in the box action described in 408(c).

To qualify as a diaphragm, a roof or floor system must be able to transmit the horizontal forces originating within its influence to the vertical resisting elements (according to their relative rigidities) without exceeding a deflection which would cause damage to any vertical element.

Diaphragms may be considered as horizontal (or slightly inclined in the case of roofs) plate girders. The slabs or flooring constitute the web; the joists, beams, or girders the stiffeners; and parts of the walls or the bond beams as the flanges. Obviously, the connections, especially along the walls, must be adequate. A long, narrow floor plan causes flexible diaphragms which may be inadequate. A rough rule is a maximum length to width ratio of 2½.

In wood roof and floor construction, diagonal sheathing or plywood panels well nailed provide much more value than square-laid sheathing. The latter generally is not considered adequate for effective diaphragm action of either roof or floor. Care must be taken to properly nail the sheathing to its periphery plates and also to reinforce around openings. Too many openings or large openings destroy the effectiveness of a floor or roof as a diaphragm.

The following was recommended in a Report of Special Committee on Horizontal Bracing Systems in Buildings Having Masonry or Concrete Walls of the Structural Engineers Association of Southern California in 1949:

$$d = \frac{75\, H^2\, F_b}{Et}$$

where d = maximum allowable deflection in inches of the horizontal bracing system of a building having masonry or concrete bearing walls and/or piers.

H = wall or pier height between horizontal supports in feet.

F_b = allowable unit compressive stress in flexure of material in the walls or piers increased ⅓.

E = modulus of elasticity of the wall or pier material.

t = wall thickness or effective pier depth in inches.

(g) Shear Walls. Vertical resisting elements of buildings may be framing, such as columns (in bending action), vertical bracing or shear walls. The latter are the most popular under modern earthquake regulations and are quite effective as well in design against blast.

A shear wall is an element of wall parallel to the direction of the assumed force which has a ratio of length to height, such that bending stresses are quite small. The wall thus has a lateral value equal to its area times the allowable unit shear. Whether the full value of the shear wall is effective in the system depends upon the distribution of forces and the relative rigidities of all the vertical elements.

(h) Layout. A great deal of a building's inherent resistance to lateral forces is determined by its basic plan layout. Desirable objectives in this regard are symmetry about both axes, not only of the building itself but of the wall openings, columns, shear walls, etc. Often it is impractical to permit the lateral force resistance to affect the utility or function of the structure and, in other cases, architectural treatment is such as to greatly increase the difficulty of the lateral force design. It is most desirable to consider the lateral forces from the start of the layout, since this may save considerable money without detracting particularly from the function or appearance of the building.

Where buildings adjoin, it is well to provide an actual separation so that they can deflect without interference or hammering. It is also desirable to separate parts of the same building from each other at abrupt changes in shape or where the type of construction changes materially. This cannot always be done. One thing is certain—either the complete separation should be made or special bracing provided to compensate for abrupt changes in plan or construction, as indicated in Fig. 4-4.

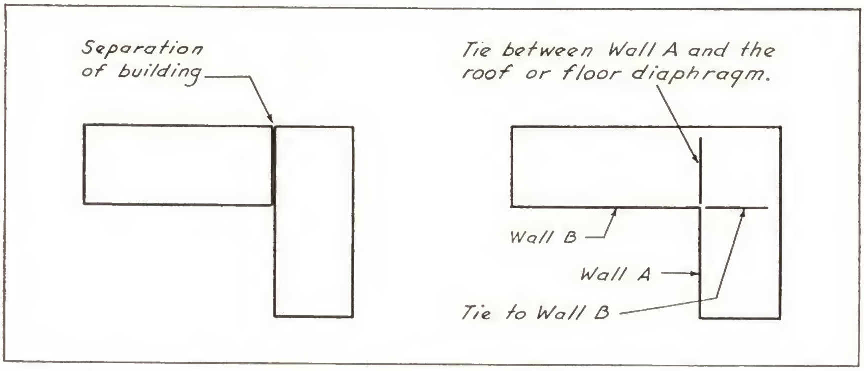


Fig. 4-4
Special Provisions for Lateral Forces Where an Abrupt Change in Plan Occurs

The amount of separation depends upon the height of the buildings involved and earthquake codes sometimes establish a minimum value. A minimum separation of ½ in. per story or 2 in., whichever is greater, is recommended.

(i) Torsion. In addition to designing the building for the controlling or critical lateral force in each direction (and only one direction and one type of lateral force need be considered at a time), it is necessary to check for and design against torsional motion, if any.

Torsion, or twisting of the building about a vertical axis, can develop with wind, earthquake or blast, and occurs simultaneously and is cumulative with one of the translational motions, particularly if the basic plan layout is not symmetrical.

Here again rigidity enters the picture. If the center of rigidity of the various vertical resisting elements in any story does not coincide with the center of gravity of the loading, torsion will result. Of course, if the difference between the two, or the eccentricity, is small, the torsional effect may be negligible. However, with an assymetrical plan layout of the vertical resisting elements, as indicated in Fig. 4-5, the effect may be considerable.

The torsional stress, delivered to any vertical element, is proportional to its contribution to the polar moment of inertia of the relative rigidities of all vertical elements per floor and also its distance from the center of rotation. The direct translational stress from the force causing the torsional moment adds algebraically to the torsional stress.

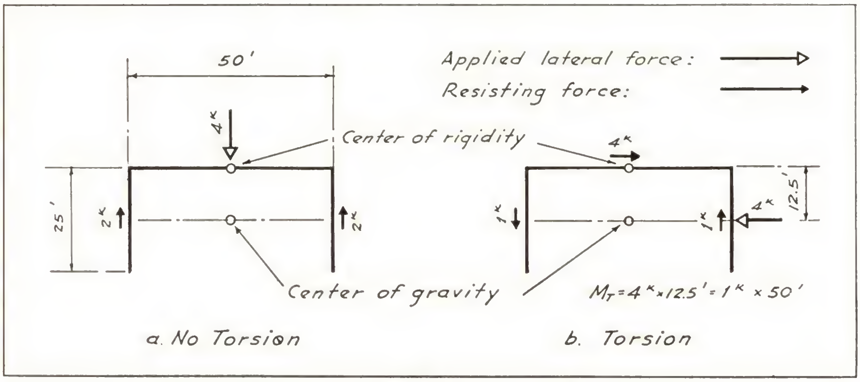


Fig. 4-5

Plan of Structure with One Open Side Illustrating Effect of Torsion

(j) Overturning. The question of overturning of the building as a whole under the action of extreme lateral forces raises some interesting considerations. The designer, providing for nominal wind forces of 15 or 20 lb. per sq. ft., naturally provides sufficient dead weight resisting moment (or anchorage) to resist the overturning moment of the wind force. In the layman's terms, the building has to weigh enough or it will blow over. Usually, a minimum safety factor of 1.5 is provided against actual overturning.

Many buildings, especially of the lighter type, have actually been "uprooted" on the windward side (if not both sides) and have blown over in hurricanes. There is some reason to believe, however, that in earthquakes and in bomb blasts the time element of uni-directional force is so short that actual overturning cannot occur, except in very unusual structures or exposures. In other words, before a building could overturn, the forces would be reduced or reversed in direction, permitting the structure to stand.

The recommended course to follow under our present state of knowledge is to provide a positive safety factor (1.5 recommended) against overturning by wind and against the assumed blast forces. For the earthquake forces, the same procedure should be followed except for very high buildings where actual overturning seems improbable.

The following, "Section 07, Provisions Against Overturning", is quoted from the proposed earthquake code of the Joint Committe of the San Francisco Section of the American Society of Civil Engineers and the Structural Engineers Association of Northern California:

"(a) The dead load moment of stability of every building or other structure shall not be less than 1½ times the overturning moment caused by wind pressure.

"(b) Provision for overturning moment shall be made for the specified earthquake forces in the top 10 stories of buildings or the top 120 ft. of other structures, and the moments shall be assumed to remain constant from these levels into the foundations."

(k) Weight. The previous paragraphs raise the question of weight or mass. From the seismic standpoint alone, the minimum weight consistent with adequate strength is the optimum condition. For this purpose, reinforced masonry of thinner section than the more conventional unreinforced "gravity-type" walls is very desirable.

In earthquakes, mass induces force and it also resists overturning of the structure as a whole. Under severe windstorm or blast forces, weight is desirable to resist overturning. Another important consideration under atomic attack is the mass of the exterior walls in resisting or preventing nuclear radiation to the contents and occupants of the building. Also, of course, is the desirability of providing incombustible materials to reduce the fire hazard resulting from any catastrophe, particularly atomic attack. Each building or locality for buildings is a problem in itself with its particular risks and basic requirements. The factors of weight or mass as well as strength, rigidity, ductility, redundancy, incombustibility, resistance to nuclear radiation and cost are all to be considered.

(l) Parts and Appendages of Buildings. In the many phases of the somewhat controversial field of seismological engineering and earthquake-resistant design, there is probably no such agreement by all concerned as there is on the subject of building parts and appendages. Engineers, who have different views on the design forces and treatment of a building as a whole, agree that it is necessary to firmly secure walls, parapet walls, architectural ornamentations, signs, canopies and all other units and appendages. Of course, they would rather not have appendages, but often they cannot be avoided.

After every major shock, the streets are littered with debris. The timing of a person running from a building and being struck by a falling piece of a parapet wall, canopy, sign, or other "appendage" is obvious. The record of such items is very bad. Modern seismic codes require a factor C (in $F=CW$) of from 0.50 to 1.00 for parts and appendages.

The proper application of such lateral forces in design should reduce this particular hazard to a negligible amount. Special consideration, however, must also be given to possible deterioration of connections and/or members from rust or other chemical action which might in time render appendages unsafe. Maintenance is very important in this regard. Los Angeles is now (1953) requiring owners of unsafe parapet walls to repair or reinforce same.

Wall panels, in addition to being tied to the frame and/or floors and transmitting their share of seismic force on the building as a whole, must also be designed as local units to resist force normal to their surfaces. Codes generally set forth a factor C of 0.20 for these forces in active earthquake regions. Thus, a wall weighing 100 lb. per sq. ft. would be subjected to a normal lateral force F of 0.20 times 100, or 20 lb. per sq. ft., which is equal to the ordinary 20 lb. per sq. ft. (sometimes 15 lb. per sq. ft.) wind force but is much less than the recommended forces for atomic bomb resistance. The important thing is that for earthquake, wind or blast, the wall and all its connections be designed to take forces in either direction, inward or outward.

(m) Connections. Another principal item about which all engineers agree, in the somewhat controversial field of earthquake-resistant design, is the absolute necessity of tying the various parts, members and elements of a building together. Every major earthquake has shown that the failure to provide ample connections and the dependence upon friction alone for any appreciable lateral force resistance lead to disaster. A building may have ample

materials and strong individual members, but if they are not so connected as to act as a unit under severe and chaotic lateral motion, together with vertical motion, they will come apart. Differential motion of parts of a building causes impact, even hammering, and failure.

A structural engineer, familiar with earthquake damage and design, considers a building not as a summation of parts, such as walls, columns, trusses, floors, roofs, etc., but as a unit that has its own properties, depending upon how the various parts are arranged and tied together.

In designing the connections or ties, it is necessary to carry out the forces and their stress paths (according to relative rigidity) and also to make each and every connection along each path adequate and consistent with the basic assumptions and distribution of forces. In designing and detailing connections, it is well to keep in mind that the lateral forces are not static, as assumed for convenience, but dynamic and to some extent unpredictable.

There is every reason to believe that the principle of properly tying or connecting the parts of a structure would be equally beneficial in resisting high wind velocities and bomb blasts. An example would be the conventional construction of a one-story commercial building of masonry walls and wood roof trusses either bearing on the walls or on pilasters in same. Where such construction has merely set the trusses on the walls or pilasters with little or no connection of the trusses to the walls (and this is very common), the earthquake record has been very bad—the trusses slide on their supports and often literally hammer the walls down. In tornadoes, the roof lifts off due to the uplift or suction forces, or the walls may "blow out" due to greater internal air pressures and lack of ties of floors and roofs to the walls.

In considering any type of lateral force, it should be the object to tie all principal parts and elements together for forces in any and all directions, including the vertical, if it is desired to have the structure remain a unit during the test. The extra cost of doing this is quite small if the designer is resourceful and the builders are not resistive to new or improved construction methods.

(n) Drawings and Construction. Providing positive and dependable resistance to high wind velocities, earthquakes or bomb blast requires, in addition to special engineering treatment and judgment, adequate design and drawings, and consistently good materials and construction. Even with these, absolute resistance or "proof" against all damage cannot be expected for very severe disasters. There is not only more to be learned about the actual phenomena and their effects on structures of all types, but modern code requirements and design methods are not expected to prevent all damage under unusually severe conditions. If either good design and drawings or good materials and construction are lacking, the results will be very unsatisfactory. Without both of these, the results may be catastrophic.

The engineering work and time involved in providing a modern lateral force-resistant structure is much more than in providing the older conventional designs with little resistance except for nominal wind forces. The fees should reflect this. No construction can be expected to be better than the design and what the drawings indicate. No structural feature or connection involved in the lateral force analysis should be omitted from the drawings and left to "the shop," or "the field", or "to supervision", or just to chance,

no matter how minor in cost. It is almost invariably true that, unless shown otherwise, construction will be in accordance with "standard practice" or conventional methods, many of which are diametrically opposed to lateral force resistance.

The materials and construction must consistently follow the specifications and the drawings. Workmanship in laying brick, placing the reinforcement, and in grouting must be good to excellent, not average. The men in the field must be made to realize and appreciate the value of good workmanship and the serious results of the opposite. The designer's responsibility is to fully indicate practical construction details, even though not conventional. The workmen, contractors, building officials, inspectors and engineers must see that these details are followed. It can be done and has been done many times. And it must be done every time and place where effective lateral force-resistant construction is desired.

CHAPTER 5

LATERAL FORCE RESISTANT STRUCTURES

501. INTRODUCTION

The preceding chapter, "Design Criteria," not only presents the various loads and forces to be provided for in the structural design of buildings but also discusses the most important design considerations with particular reference to lateral forces. This chapter will present some examples of a lateral force design procedure for simple masonry structures. It is hoped that the reader will be thoroughly familiar with or will refer to Design Considerations, Section 408, before following these design examples, and that he realize no lateral force design method or design assumption is necessarily applicable to all structures nor intended to be more than an aid to good engineering judgment.

502. GENERAL CONSIDERATIONS

In the layout of masonry bulidings, the designer must not only think of the interior space requirements and allocations but he should also keep in mind the horizontal and vertical dimensions of the brick when laid. The location and spacing of wall openings can be executed so as to provide masonry piers which are a function of laid brick lengths, preferably in keeping with modular dimensions, and also the maximum resistance to lateral forces. Every section of a wall is an important element in resistance to earthquake, wind or blast. Architects, engineers, designers, building inspectors, contractors, and the workmen on the jobs must realize this for the best results. The walls have much more to do than hold the roof up, keep the occupants in, and the weather out.

The examples to follow are not actual buildings which have been constructed but hypothetical cases selected to emphasize various factors. The structures are not intended to be used for any other purpose than as aids in illustrating certain design methods.

The codes, forces and unit stresses used in the various examples are also for illustrative purposes only. Chapter 4 presents various load and force criteria and recommendations, and Chapter 3 sets forth design unit stresses and factors for various conditions.

503. ALLOWABLE UNIT STRESSES

The following assumptions as to materials and working stresses will be made in the lateral force computations for reinforced brick masonry (see Chapter 3):

Compressive strength of brick units = 2500 psi
Mortar: Type A-1 (See Section 203)
f'_m = 60 per cent × 2500 = 1500 psi

	Stress	Allowable Value (psi)	
f_m	Compression, axial	0.20 f'_m	300
f_m	Compression, Flexural	0.33 f'_m	500
v_m	Shear, no web reinforcement		50
v	Shear, with web reinforcement		150
f_m	Bearing	0.25 f'_m	375
E_m	Modulus of elasticity (1)	1000 f'_m	1.5×10^6
E_v	Modulus of rigidity (in shear) (1)	400 f'_m	0.6×10^6
u	Bond, plain bars		80
u	Bond, deformed bars (ASTM A305-)		160
f_s	Tensile unit stress in steel reinforcing bars		20,000

(1) E_m = 1,000,000 psi has been assumed in determining pier rigidities in Figs. 4-2, 4-3, 5-1, and in the design examples. This was for convenience only and has no effect on the relative rigidities so long as masonry is not compared in rigidity with other materials. If such should be done, the correct values of E_m and E_v for all the materials should be used.

Note: The above stresses may be increased 33⅓ per cent for resisting lateral forces alone or in combination with other loads provided the basic allowable values are not exceeded for dead load and live load alone.

504. SYMBOLS AND ABBREVIATIONS FOR LATERAL FORCE DESIGN

The following symbols and abbreviations will be used in the lateral force computations:

t	Wall or pier thickness
h	Pier height
d	Pier width, total
d_s	Pier width, to tensile steel (d − 2 in.)
R_1	Rigidity of a single pier
R_Σ	ΣR_1 of piers of same type in one wall
R	Relative rigidity of piers of same type in one wall
P	Lateral force taken by a single pier
ΣP	Lateral force taken by piers of same type in one wall
H	Lateral force taken by one wall
ΣH	Total horizontal force applied to a building story
A	Area of pier (t × d_s, or t × d if unreinforced)
M	Moment in pier due to lateral force
S	Section modulus of pier (1/6 td_s^2, or 1/6 td^2 if unreinforced)
F_b	Actual flexural stress in pier (M/S)
F_o	Actual axial stress in pier due to M_o
M_o	Overturning moment due to H
RM	Resisting moment (resists M_o)
F_a	Actual axial compressive stress in pier due to vertical D.L. and L.L
D.L.	Dead load

L.L. Live load
ΣF Maximum total unit compressive stress in pier ($F_b + F_o + F_a$)
v Shear stress (3/2 $\times$ P/A, unreinforced) (8/7 $\times$ P/A, reinforced)
u Bond stress
W Total vertical D.L. and L.L.

Note: Units; inches or pounds as applicable.

Fig. 5-1 is a graph giving the relationship of pier rigidity, R_1, to the ratio of h/d for piers fixed at both ends and fixed at one end only (cantilever). The derivation of the equations is given with Fig. 4-2, page 148.

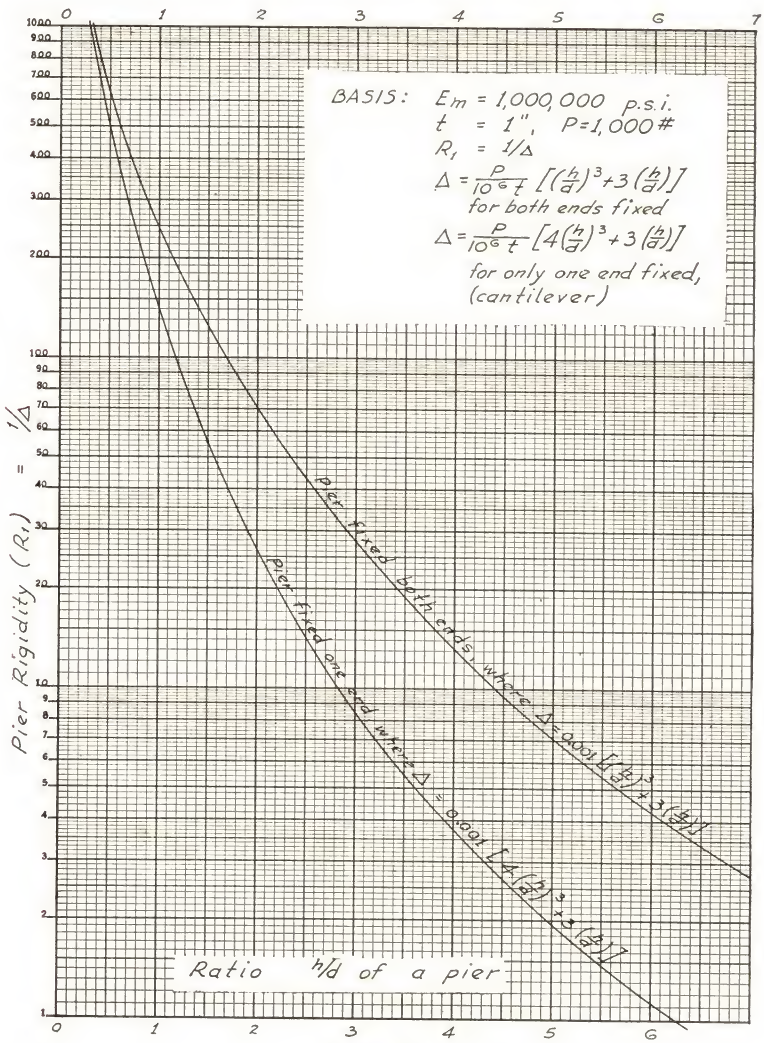


Fig. 5-1
Pier Rigidities

505. SINGLE-FAMILY ONE-STORY RESIDENCE—UNREINFORCED "SCR brick" *

The floor plan and wall elevations, Fig. 5-2, represent a hypothetical masonry bearing wall, one-story residence. In order to simplify this first example, the functional efficiency of the layout is sacrificed to provide wall openings symmetrical about both axes of the building, thus eliminating the torsional "twist" of the lateral forces. An excessive number of openings is provided to illustrate the mechanics of the calculation. While the analysis may indicate a satisfactory design, sound engineering judgment would dictate the use of fewer openings (more brick pier area) for a lateral force-resistant structure. The calculations are not precise—to make them so would be impractical. They should represent a minimum design basis and a springboard to good judgment and design sense.

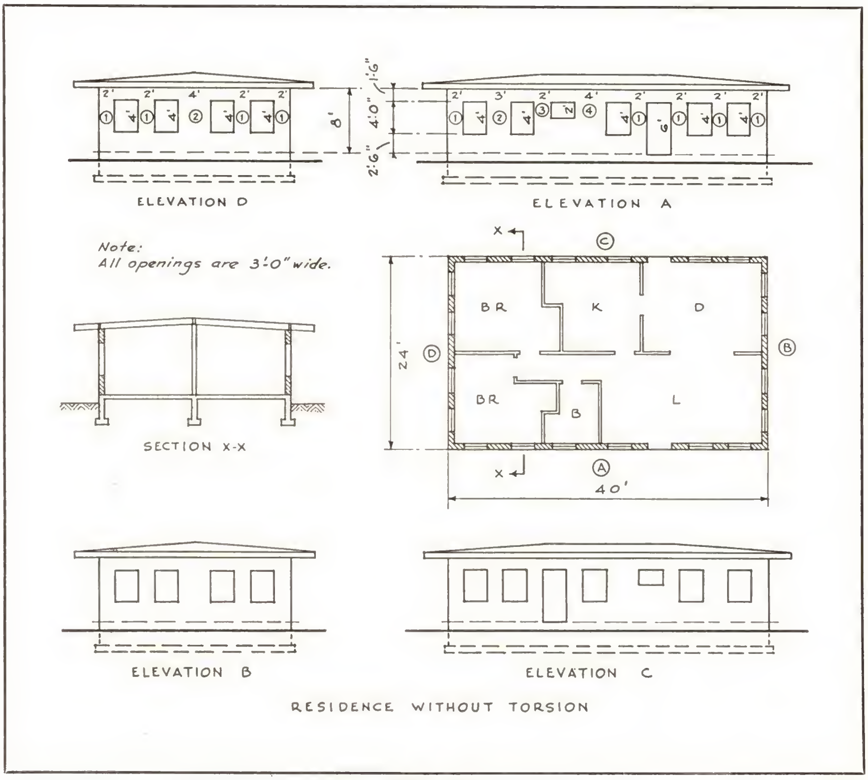


Fig. 5-2
One-Story Residence

This "building" will be analyzed first with assumed unreinforced 6-in. "SCR brick" walls and then with 9-in. reinforced brick masonry walls. The 1952 Uniform Building Code lateral force design requirements for Zone 3 will be used as an earthquake criterion in these examples. The resulting 13.3 per cent seismic force is greater than for other codes or recommendations. Comparisons for wind and blast forces will also be presented.

* Reg. U. S. Pat. Off., Pat. Pend., SCPRF.

Table 5-1 is for the nominal 6-in. "SCR brick" walls. Note that only one wall is used for each assumed direction due to the symmetry of wall openings, but that only half the total seismic force is used on each wall. The various piers in each wall are assigned reference numbers and their t, h, d and h/d values are entered in the table. This tabular form and method of analysis are for "box action" as described in Section 408(c) and 408(d). The roof construction must therefore qualify as an effective diaphragm as indicated in Section 408(f).

TABLE 5-1

"SCR brick" 1952 U.B.C. EARTHQUAKE FOR ZONE 3

Pier	No.	t in.	h in.	d in.	h/d	R_1	R_Σ	R	H lb.	ΣP lb.
A–1	5	5½	48	24	2	70	350	.26	....	740
–2	1	5½	48	36	1.33	160	160	.12	....	340
–3	1	5½	24	24	1	250	250	.18	....	510
–4	1	5½	24	48	0.50	610	610	.44	....	1,260
							1,370	1.00	2,850	2,850
B–1	4	5½	48	24	2	70	280	.53	....	1,510
–2	1	5½	48	48	1	250	250	.47	....	1,340
							530	1.00	2,850	2,850

"SCR brick" 1952 U.B.C. EARTHQUAKE FOR ZONE 3 (Continued)

Pier	P lb.	A sq. in.	v psi	M in.-lb. x 10^{-3}	S in.³ x 10^{-3}	F_b psi	F_o psi	F_a psi	ΣF psi
A–1	150	132	1.7	3.6	0.53	6.8	1.5	12.7	21.0
–2	340	198	2.6	8.2	1.18	7.0	1.0	12.7	20.7
–3	510	132	5.8	6.1	0.53	11.5	....	12.7	24.2
–4	1,260	264	7.2	15.1	2.12	7.1	....	12.7	19.8
B–1	380	132	4.3	9.1	0.53	17.2	3.8	12.7	33.7
–2	1,340	264	7.6	32.2	2.12	15.2	....	12.7	27.9

Determination of Lateral Force to be Applied
Earthquake:

Dead load down to mid-height of wall:

Roof—24 ft. × 40 ft. × 15 lb. per sq. ft. (Weight prorated to include eave overhang) = 14,400 lb.

Walls—4 ft. × 128 ft. × 60 lb. per sq. ft. × 75 per cent (Approximately 25 per cent openings) = 23,100

Partitions—90 ft. × 4 ft. × 15 lb. per sq. ft. = 5,400

42,900 = W

From 1952 U.B.C. (Zone 3) : $\Sigma H = CW$

Where W = total dead load

$$C = \frac{.15}{N + 4.5} \times 4$$

N = number of stories above the story under consideration

$$C = \frac{.15}{0 + 4.5} \times 4 = 0.133$$

$$\Sigma H = .133 \times 42{,}900 \text{ lb.} = 5{,}700 \text{ lb.}$$

Distribution of total lateral force:

Because of the symmetry of the building,

$$H_A = H_C = \tfrac{1}{2} \times 5{,}700 \text{ lb.} = 2{,}850 \text{ lb.}$$
$$H_B = H_D = \tfrac{1}{2} \times 5{,}700 \text{ lb.} = 2{,}850 \text{ lb.}$$

The rigidity of each pier (R_1) is obtained from Fig. 5-1 by locating on the proper curve points which correspond to the h/d value for each pier. Rigidity is assumed at each end of each pier and will be verified later in the calculations. The academic-minded reader will question the fact that the upper element which is assumed to fix the piers is only 18 in. deep, whereas the piers vary in width from 24 to 48 in. Obviously, even though the 18-in. wall section may have the strength, its flexural rigidity in proportion to the piers cannot be constant. Moreover, end piers have less restraint than interior piers. A more rigorous approach could be made, and should be in unusual cases, but in view of the uncertainties of the earth motion and other factors, this refinement is not warranted here. The illustrated method, plus the use of good judgment, will suffice in most cases for simple structures.

The column headed R_Σ is the product of the number of piers (of each type) and their rigidities. The relative rigidity (R) is the proportion that each R_Σ is of the total. The relative rigidity is the measure of the amount of total horizontal force (H) carried by each type of pier.

The calculation of H follows Table 5-1. First, the approximate total dead load of the roof and ceiling is assumed to average 15 lb. per sq. ft. over the area of the building (a more accurate method would be to take the entire roofed area at one unit weight and then add the ceiling weight). The walls and the partitions are assumed to contribute half their height, or 4 ft., to the seismic force (ΣH), the other half being delivered directly to the ground. This is, of course, an approximation like the assumption that 25 per cent of the wall area is in openings. Such approximations should be reasonable and should be consistently maintained throughout the work for similar conditions.

The total seismic force (ΣH) equals 13.3 per cent of the total seismic weight (W). As previously stated, half of ΣH goes to each wall (each direction is taken successively) because of the symmetry. H is distributed to the various types of piers in proportion to R and then the force to each pier is obtained by dividing ΣP by the number of piers of each type. This force (P) produces bending moment and shear in each pier. The balance of Table 5-1 is devoted to determining the maximum unit shears, bending and axial unit stresses.

The "SCR brick" section is considered as a solid in spite of the cores (holes), since the cores are a small (25 or less) percentage of the total area and are located away from the faces where greater bending stresses occur. Thus, A is the product of t, the net width of 5½ in. by the length d. The maximum unit shear is taken as $3/2 \times P/A$ to allow for the variation in shear on a homogeneous section. This factor, 1.5, may be considered slightly conservative by some designers. The shearing stresses are tabulated in the

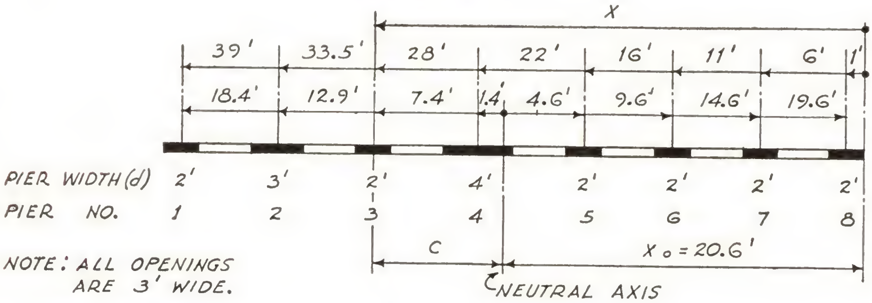
column headed v and it will be noted that the unit shears are greater for the wide, more rigid piers. The moment calculation is based upon each pier being fixed top and bottom with the horizontal force applied at the top; thus, $M = Ph/2$. The section modulus, S, $= td^2/6$ and $F_b = M/S$.

In addition to the bending stress due to the horizontal forces on the piers, there is a tendency toward axial stress due to overturning moment of the building as a whole (cantilever moment). Although this is obviously quite small in this low structure, it is calculated in Table 5-2 and the results are entered in Table 5-1 in the column headed F_o. The total overturning moment is obtained by multiplying the seismic force (H) per wall by a height to the bottom of the piers. This height has been taken here as 5 ft., representing an average (and approximate) distance of the center of gravity of the contributing masses about the bottom of the piers (the window sill levels). This level is taken since it will be the critical point for both axial and bending stresses in the piers.

TABLE 5-2

DETERMINATION OF OVERTURNING STRESS

Walls A and C



Note: Wall returns neglected for simplicity.

Overturning Moment: $M_o = 2{,}850$ lb. x 5 ft. $= 14{,}200$ ft.-lb.

To locate neutral axis of wall: $x_o = \dfrac{\Sigma dtx}{\Sigma dt} = \dfrac{\Sigma dx}{\Sigma d}$

Pier No.	d ft.	x ft.	dx ft.^{2}
8	2	1	2
7	2	6	12
6	2	11	22
5	2	16	32
4	4	22	88
3	2	28	56
2	3	33.5	100.5
1	2	39	78
	19		390.5

$$x_o = \frac{390.5}{19} = 20.6 \text{ ft.}$$

To compute Moment of Inertia of wall:

$$I_o = \Sigma dtc^2 = t\Sigma dc^2$$

Where the moment of inertia of a pier about its own axis is very small, it is neglected and the pier area is considered concentrated at its neutral axis.

Pier No.	d ft.	c ft.	dc^2 $ft.^3$	F_o psi
8	2	19.6	770	1.5
7	2	14.6	426	1.2
6	2	9.6	184	
5	2	4.6	42	
4	4	1.4	8	
3	2	7.4	110	
2	3	12.9	500	1.0
1	2	18.4	678	1.5
			2,718	

$$t = \frac{5.5}{12} \text{ ft.}$$

$$I_o = 2{,}718 \text{ x } \frac{5.5}{12} = 1{,}250 \text{ ft.}^4$$

To compute stress in pier F_o:

$$F_o = \frac{M_o c}{I_o} = \frac{14{,}200}{1{,}250} \text{ x c lb. per sq. ft.}$$

$$= \frac{14{,}200}{1{,}250 \text{ x } 144} \text{ x c psi}$$

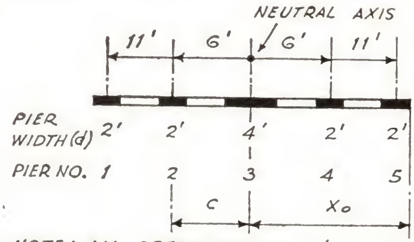
$$= .079 \text{ c psi}$$

Walls B and D

Overturning Moment: $M_o = 2{,}850$ lb. x 5 ft. = 14,200 ft.-lb.

Neutral axis through center from symmetry
To compute Moment of Intertia of wall:

$$I_o = \Sigma dtc^2 = t\Sigma dc^2$$



NOTE: ALL OPENINGS ARE 3' WIDE.

Note: Wall returns neglected for simplicity

Pier No.	d ft.	c ft.	dc^2 $ft.^3$	F_o psi
1	2	11	242	3.8
2	2	6	72	2.1
3	4	0	0	0
4	2	6	72	2.1
5	2	11	242	3.8
			628	

To compute stress in pier F_o:

$$F_o = \frac{14{,}200}{287 \text{ x } 144} \text{ x c psi}$$

$$= .346 \text{ c psi}$$

$$t = \frac{5.5}{12} \text{ ft.}$$

$$I_o = 628 \text{ x } \frac{5.5}{12} = 287 \text{ ft.}^4$$

less than the required wind force, the latter would be used. The seismic force applied to an individual element, such as the wall under consideration, is not combined with the force on the structure as a whole. The general case and the local loading constitute two separate considerations; whichever produces the greater unit stresses governs the design. In practice, the wall should be cleared for local (normal) loading before proceeding to the analysis of the structure as a whole.

The above has been based entirely upon earthquake design. The earthquake force (ΣH) of 5700 lb. may be compared with wind pressure on the building as a whole as follows:

Allow 1 ft. for the equivalent area of the eaves, roof pitch, drag, etc., above the 8 ft. of wall. Then on the narrow end of the building the equivalent wind force would be approximately $\frac{5700}{(1 + 4)\,24} = 47.5$ lb. per sq. ft.

On the wide side of the building, the equivalent wind force would be $\frac{5700}{(1 + 4)\,40} = 28.5$ lb. per sq. ft. These values are not applicable unless the walls can withstand such forces normal to their surfaces; i.e., in local bending. This will be calculated for a typical 2-ft. wide pier:

$$S = \frac{5.5^2 \times 24}{6} = 121 \text{ in.}^3$$

Allowable flexural tension	$= 26.7 = (20 \times 1\tfrac{1}{3})$
Add average compression	$= \underline{12.7}$
	39.4 psi
Moment	$= 39.4 \times 121 = 4760$ in.-lb.

Assuming a vertical beam, fixed at the base and pinned at the top, the maximum moment in the pier area (between openings) would be:

$$M = \frac{9\ Fl}{128} = 4760 \text{ in.-lb.}$$

$$F = \frac{128 \times 4760}{9 \times 8 \times 12} = 705 \text{ lb. total force allowable.}$$

The moment at the base is greater but there is more width for moment resistance; therefore, the above value will be considered critical.

If the pier must resist wind on 5 ft. of width, 8 ft. high, the allowable unit pressure $= \frac{705}{5 \times 8} = 17.6$ lb. per sq. ft. Thus, with the assumption that the windows do not fail first, the building could withstand an approximate wind force of 18 lb. per sq. ft. We thus find that the excess amount of wall openings causes some concern with the wind forces. Judgment would dictate that the area of window openings be decreased or the wall height reduced. This is a good example of the importance of checking all factors in lateral force design and the importance of not weakening a structure too much with excessive wall openings. A reinforced or even partially reinforced wall construction would greatly improve the inherent resistance of the structure to lateral forces.

506. SINGLE-FAMILY ONE-STORY RESIDENCE—9-IN. REINFORCED BRICK MASONRY

The same plan and wall elevations (Fig. 5-2) will now be calculated with 9-in. brick walls reinforced as indicated in Fig. 5-4, in lieu of the nominal 6-in. unreinforced "SCR brick" of the previous example.

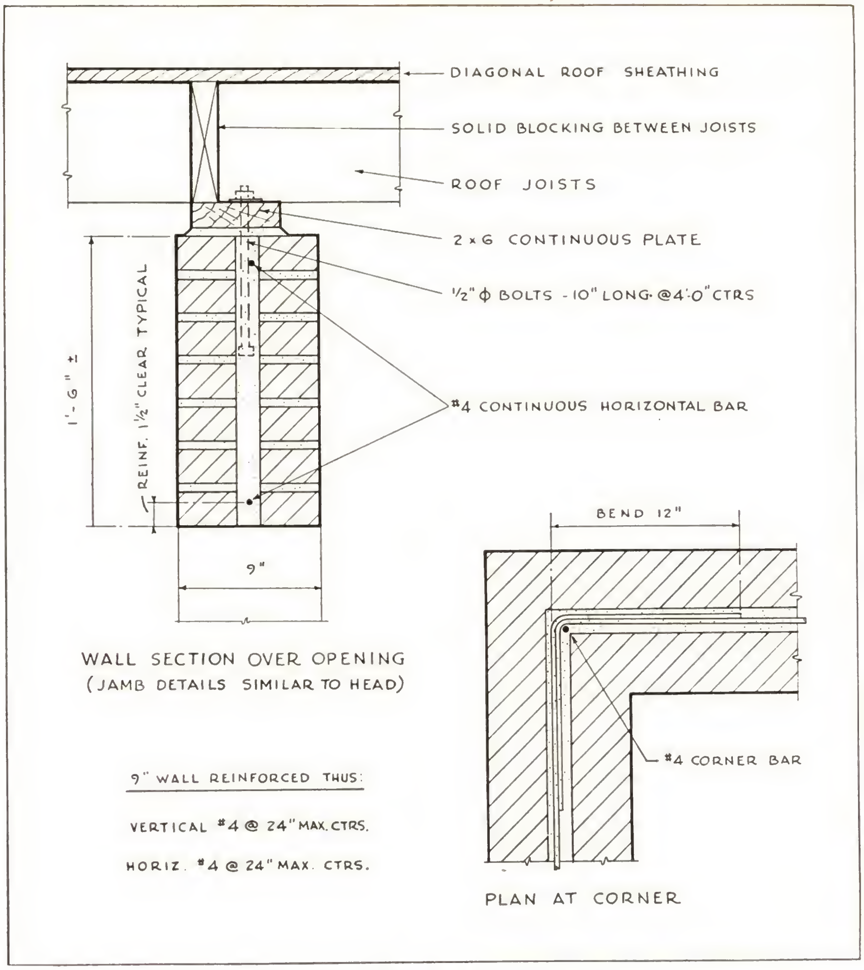


Fig. 5-4
9-in. RBM Wall Details

The tabulated calculations are shown in Table 5-4.

The analysis is the same as for the "SCR brick" construction, except for the thicker walls and the fact that this masonry is reinforced. The thicker walls increase value (t) to 9 in. and the weight and seismic force values must be increased because of the heavier walls. The reinforcement, which is assumed to take all tension, alters the calculation of A, v, and S. As noted in Section 504, $A = td_s$ where reinforcement is used and $S = td_s^2/6$. The distance from the compression face to the center line of the vertical pier reinforcement (d_s) is taken as d — 2 in. The unit shear is calculated as $v = P/jd_st$, and j is assumed as ⅞; thus, $v = 8P/7A$.

In determining rigidities, however, the full pier width (d) is used as for the unreinforced masonry. This is approximate but simple in application and tends to allow for the extra rigidity provided by the reinforcement.

The analysis is completed the same as for the "SCR brick" wall. However, the F_b values, calculated with the section modulus, are for the compressive stress only; these are approximate values but are close enough for the intended purpose.

TABLE 5-4

9-IN. RBM WALLS

1952 U.B.C. EARTHQUAKE FOR ZONE 3

Pier	No.	t in.	h in.	d in.	h/d	R_1	R_Σ	R	H lb.	ΣP lb.
A-1	5	9	48	24	2	70	350	.26		980
-2	1	9	48	36	1.33	160	160	.12		450
-3	1	9	24	24	1	250	250	.18		670
-4	1	9	24	48	0.50	610	610	.44		1,650
							1,370	1.00	3,750	3,750
B-1	4	9	48	24	2	70	280	.53		2,000
-2	1	9	48	48	1	250	250	.47		1,750
							530	1.00	3,750	3,750

9-IN. RBM WALLS

1952 U.B.C. EARTHQUAKE FOR ZONE 3 (Continued)

Pier	P lb.	A sq. in.	v psi	M in.-lb. x 10^{-3}	S in.^{3} x 10^{-3}	F_b psi	F_o psi	F_a psi	ΣF psi
A-1	196	198	1.1	4.7	0.72	6.5	1.2	11.2	18.9
-2	450	306	1.7	10.8	1.73	6.3	0.7	11.2	18.2
-3	670	198	3.9	8.1	0.72	11.3	....	11.2	22.5
-4	1,650	414	4.6	19.9	3.17	6.3	....	11.2	17.5
B-1	500	198	2.9	12.0	0.72	16.7	3.1	11.2	31.0
-2	1,750	414	4.8	42.1	3.17	13.3	....	11.2	24.5

Determination of Lateral Force to be Applied

Earthquake

Dead load down to mid-height of wall:

Roof—24 ft. × 40 ft. × 15 lb. per sq. ft. = 14,400 lb.
(Weight prorated to include eave overhang)

Walls—4 ft. × 128 ft. × 95 lb. per sq. ft. × 75 per cent = 36,500
(Approximately 25 per cent openings)

Partitions—90 ft. × 4 ft. × 15 lb. per sq. ft. = 5,400

56,300 lb. = W

$\Sigma H = .133 \times 56{,}300 \text{ lb.} = 7{,}500 \text{ lb.}$

$H_A = H_B = H_C = H_D = \frac{1}{2} \times 7{,}500 \text{ lb.} = 3{,}750 \text{ lb.}$

Since the plan and elevations of this structure are the same as the "SCR brick" building, the stresses due to overturning moment will be directly proportional to the lateral force (H) and inversely proportional to the wall thickness (t); that is, $F_o = \frac{5.5}{9} \times \frac{3750}{2850} \times F'_o = .81\ F'_o$ where F'_o is the value for "SCR brick" given in Table 5-1. The axial compressive stress in piers is determined as follows:

Load on typical 2-ft. wide pier

Roof D.L.	5 ft. × 10 ft. × 15 lb. per sq. ft.	=	750 lb.
Wall	5 ft. × 1.5 ft. × 95 lb. per sq. ft.	=	710
	2 ft. × 4 ft. × 95 lb. per sq. ft.	=	760
			2220 lb.

$$F_a = \frac{2220 \text{ lb.}}{198} = 11.2 \text{ psi.}$$

This value is used as an average maximum value.

The tension is, of course, taken by the reinforcing bars. The allowable unit stresses are given in Section 503, and the following are the critical stresses:

Critical Stresses

Shear

Actual = 4.8 psi in pier B-2

Allowable = $50 \times 1\frac{1}{3} = 67$ psi.

Compression

Actual = 31.0 in pier B-1

Allowable axial ① = $300 \times 1\frac{1}{3} = 400$ psi.

Tension in Pier

Actual moment = 42.1 in.–kips.

Allowable moment = $A_s f_s jd \doteq .20 \times 20{,}000 \times 1\frac{1}{3} \times \frac{7}{8} \times (48-2)$

= 214 in.–kips

Actual Steel = .20 sq. in.

Allowable A_s $= \frac{M}{f_s jd} = \frac{42.1 \text{ (pier B-2)}}{(20 \times 1\frac{1}{3}) \times \frac{7}{8} \times (48-2)} = .04$ sq. in.

① Because actual stress is so low, the allowable combined axial and flexural stress has not been calculated.

The strength of the piers in shear, compression, and in tension (taken by No. 4 bars) is far greater than necessary to meet the 13.3 per cent seismic factor. The spandrel will have a maximum moment (under the assumed conditions) of one-half of 42.1 or about 21.1 in.-kips at piers B-2 and D-2. A No. 4 bar placed as shown in Fig. 5-4 would be stressed as follows from the seismic forces: $f_s = \frac{M}{A_s jd_s} = \frac{21{,}100}{0.20 \times \frac{7}{8} \times 16} = 7500$ psi, which leaves excess capacity for nominal beam loading from the window spans.

The seismic force normal to the wall would be $0.20 \times 95 = 19$ lb. per sq ft. less than the 20 lb. per sq. ft. wind force.

Blast

Since there is so much excess capacity for the 13.3 per cent seismic factor, the walls of the same structure will be tried for blast resistance. The roof construction, sash, eave overhang, and other non-masonry features will not be analyzed here since they are beyond the scope of this book but, of course, should be carefully designed in actual practice. It will be assumed here that all such items are adequate.

TABLE 5-5

BLAST EFFECTS

On Walls Parallel to Direction of Blast (Blast on 40-ft. side of building)

Note: One foot added to half the wall height to represent equivalent area for eave, roof pitch, drag, etc.

ΣH for blast $= (1 + 4)$ ft. x 40 ft. x 90 lb. per sq. ft. $= 18{,}000$ lb.

Blast stresses will be $\dfrac{18{,}000}{7{,}500} = 2.4$ times those due to 13.3 per cent earthquake

Shear $= 2.40 \times 4.8 = 11.5$ psi

Compression $= 2.40 \times 31.0 = 74.4$ psi

Area of steel $= 2.40 \times 0.04 = 0.10$ sq. in.

Above stresses are less than allowable and steel requirement is less than actual.

Spandrel

The maximum moment due to lateral forces to which the spandrel will be subjected $= \frac{1}{2} \times 42.1 \times 2.4 = 50.5$ in.-kips

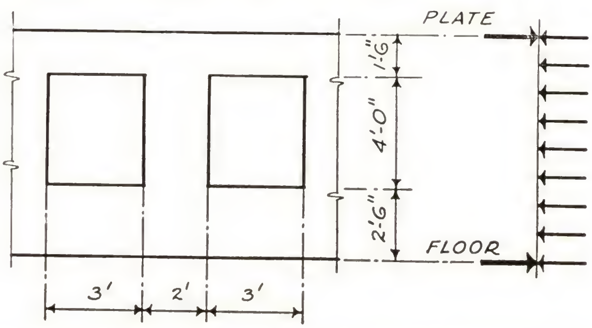
$$\text{Actual } f_s = \frac{M}{A_s\, jd} = \frac{50{,}500}{0.20 \times 7/8 \times (18-2)} = 18{,}000 \text{ psi}$$

Allowable $f_s = 20{,}000 \times 1\frac{1}{3} = 26{,}700$ psi

$$\text{Actual } f_m = \frac{M}{1/6\, bd^2} = \frac{50{,}500}{1/6 \times 9 \times (18-2)^2} = 132 \text{ psi}$$

Allowable $f_m = 500 \times 1\frac{1}{3} = 667$ psi

On Walls Normal to Direction of Blast (*See diagram*)



Note: In an actual design, window areas which will offer no resistance to blast forces need not be included in the tributary area.

$w = 90$ lb. per sq. ft. x 5 ft. $= 450$ lb. per ft.

$W = 450$ lb. per ft. x 8 ft. $= 3600$ lb.

$M = \frac{1}{8} \times 3{,}600 \times 8 \times 12 = 43{,}200$ in.-lb.

$$\text{Actual } f_m = \frac{M}{1/6\ bd_s^2} \text{ (approximate)} = \frac{43{,}200}{1/6 \times 24 \times (4.5)^2} = 533 \text{ psi}$$

Allowable $f_m = 500 \times 1\frac{1}{3} = 667$ psi

(Note that F_a is neglected because it is very small.)

$$A_s = \frac{M}{f_s\, jd} = \frac{43{,}200}{20{,}000 \times 1\frac{1}{3} \times \frac{7}{8} \times 4.5} = 0.41 \text{ sq. in. required}$$

A_s furnished (No. 4 each jamb) $= 2 \times .20 = .40$ sq. in.

Therefore, the original design is satisfactory for recommended blast pressures.

Calculations in Table 5-5 compare the seismic results to the effects of horizontal blast loading as recommended by the Federal Civil Defense Administration (refer to Section 407). With the horizontal pressure of 90 lb. per sq. ft., the 9-in. brick walls with nominal reinforcement (see Section 308(h)5) are not overstressed. Additional reinforcement could be provided at strategic points to further increase the resistance of the structure.

507. TWO-STORY RESIDENCE

Fig. 5-5 is a two-story residence with 9-in. reinforced brick masonry bearing walls reinforced as shown in Fig. 5-4, page 167.

As for the one-story residence, this hypothetical structure has been developed to illustrate various features of lateral force design. No attempt has been made to indicate a building necessarily desirable in either function or appearance.

The top story will not be analyzed. The lower story, which is critical with the same wall construction in both stories, has wall openings located so as to cause torsional stress. Since the rigidities of the walls are not equal, the center of mass will not coincide with the center of rotation. The resulting tendency to "twist" must be resisted by the masonry piers. The resistance to this twist or torsion adds to the forces carried by some of the brick piers and deducts from others. The analysis of this action can be made in various ways. In general, a polar moment of inertia of the rigidities is obtained and the amount of the torsional moment is distributed to each element in proportion to its rigidity and distance from the center of rotation. It is not necessary with ordinary wall-bearing structures to consider the resistance of the masonry piers normal to their faces for either torsional or translational motion since the rigidity in this direction is usually a small part of that parallel to the wall.

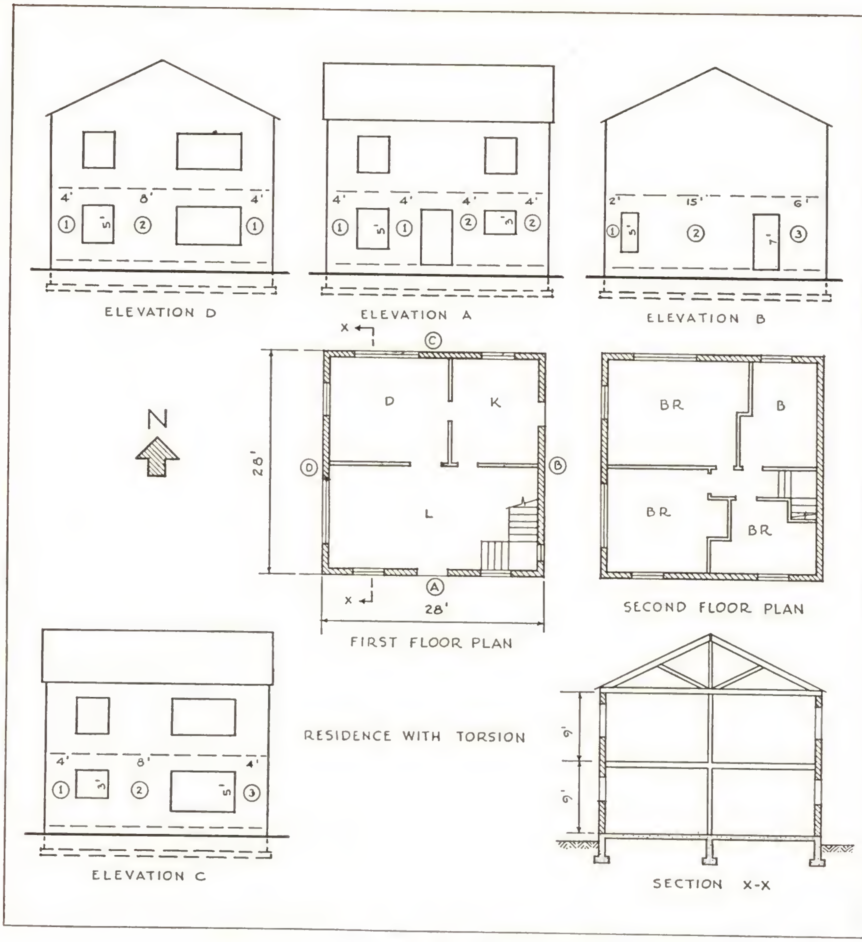


Fig. 5-5
Two-Story Residence

Table 5-6 illustrates the tabular analysis of the lower story of the structure. Because of the asymmetry of the wall rigidities, two walls are considered for the translational motion in each direction. For example, walls A and C are considered to resist the force in the east-west direction. However, in determining the force taken by each wall in resisting the torsional moment, all four walls are considered. Table 5-7 indicates this process. Rigidities of each wall as a whole are used to determine the center of rigidity or of rotation, CR. The center of gravity is assumed here to be at the center of the building. The torsional moment is the seismic force times the "eccentricity" or distance between the center of rotation and the center of gravity or mass. Obviously, if there were no eccentricity, the torsional moment would be zero; this is desirable but often difficult to obtain.

TABLE 5-6

9-IN. RBM WALLS — ANALYSIS FOR FIRST STORY

1952 U.B.C. EARTHQUAKE FOR ZONE 3

Pier	No.	t in.	h in.	d in.	h/d	R_1	R_Σ	R	H lb.	ΣP lb.
A-1	2	9	60	48	1.25	175	350	.32		3,000
-2	2	9	36	48	0.75	375	750	.68		6,400
							1,100	1.00	9,400	9,400
C-1	1	9	36	48	0.75	375	375	.27		2,780
-2	1	9	36	96	0.375	840	840	.60		6,180
-3	1	9	60	48	1.25	175	175	.13		1,340
							1,390	1.00	10,300	10,300
B-1	1	9	60	24	2.50	43	43	.03		320
-2	1	9	60	180	0.33	950	950	.80		8,560
-3	1	9	84	72	1.165	200	200	.17		1,820
							1,193	1.00	10,700	10,700
D-1	2	9	60	48	1.25	175	350	.42		3,740
-2	1	9	60	96	0.625	475	475	.58		5,160
							825	1.00	8,900	8,900

9-IN. RBM WALLS—ANALYSIS FOR FIRST STORY

1952 U.B.C. EARTHQUAKE FOR ZONE 3 (Continued)

Pier	P lb.	A sq. in.	v psi	M in.-lb. x 10^{-3}	S in.^{3} x 10^{-3}	F_b psi	F_o psi	F_a psi	ΣF psi
A-1	1,500	414	4.2	45.0	3.17	14.2	13.3	18.6	46.1
-2	3,200	414	8.8	57.6	3.17	18.2	13.3	18.6	50.1
C-1	2,780	414	7.7	50.0	3.17	15.7	14.4	18.6	48.7
-2	6,180	846	8.4	111.0	13.23	8.4	5.6	18.6	32.6
-3	1,340	414	3.7	40.1	3.17	12.6	16.7	18.6	47.9
B-1	320	198	1.8	9.6	0.72	13.3	13.6	18.6	45.5
-2	8,560	1,602	6.1	257.0	47.5	5.4	9.8	18.6	33.8
-3	1,820	630	3.3	76.5	7.35	10.4	13.5	18.6	42.5
D-1	1,870	414	5.2	56.2	3.17	17.7	14.4	18.6	50.7
-2	5,160	846	7.0	155.0	13.23	11.7	4.8	18.6	35.1

Determination of Lateral Force to be Applied

Earthquake

Dead load down to mid-height of lower story walls

				$\times$ .109
Roof	28 ft. $\times$ 28 ft. $\times$ 15 lb. per sq. ft.	=	11,800 lb.	1,280 lb.
2nd Floor	28 ft. $\times$ 28 ft. $\times$ 15 lb. per sq. ft.	=	11,800	1,280
Partitions	28 ft. $\times$ 28 ft. $\times$10 lb. per sq. ft.	=	7,900	860
Walls	2 $\times$ 28 ft. $\times$ 15.5 ft. $\times$ 95 lb. per sq. ft. $\times$ 80 per cent	=	65,800	7,170
Walls	2 $\times$ 28 ft. $\times$ 19.5 ft. $\times$ 95 lb. per sq. ft. $\times$ 80 per cent	=	82,800	9,030
			180,100 lb.	19,620 lb.

Note: Heights of walls taken for weights are approximate.

From 1952 U.B.C. (Zone 3) $\Sigma H = CW$

$$C = \frac{.15}{N + 4.5} \times 4$$

$$C = \frac{.15}{1 + 4.5} \times 4 = .109$$

$\Sigma H = .109 \times 180{,}100$ lb. $= 19{,}600$ lb.

Determination of Height of Application of Lateral Force

1.28 kips	$\times$ 22 ft.	=	28.2 ft.-kips
1.28	$\times$ 8	=	10.2
.86	$\times$ 14	=	12.0
7.17	$\times$ 12	=	86.2
9.03	$\times$ 16	=	144.5
19.62 kips			281.1 ft.-kips

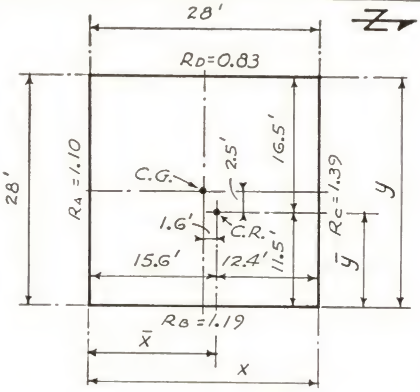
Moment arm $= \dfrac{281.1}{19.62} = 14.3$ ft.

TABLE 5-7

DISTRIBUTION OF LATERAL FORCES

$$\bar{y} = \frac{\Sigma Ry}{\Sigma R} = \frac{0.83 \times 28}{0.83 + 1.19} = 11.5 \text{ ft.}$$

$$\bar{x} = \frac{\Sigma Rx}{\Sigma R} = \frac{1.39 \times 28}{1.39 + 1.10} = 15.6 \text{ ft.}$$



$M_{N\text{-}S} = 19.6$ kips x 2.5 ft. $= 49.2$ ft.-kips

$M_{E\text{-}W} = 19.6$ kips x 1.6 ft. $= 31.4$ ft.-kips

Moment of Inertia of Walls About Point of Rotation

Wall	RΣ (a)	c	c^2	ac^2
A	1.10	15.6	243	268
C	1.39	12.4	154	214
B	1.19	11.5	132	157
D	0.83	16.5	272	226
				$I_R = 865$

Note that "(a)" is in terms of the wall rigidity = RΣ (from Table 5-6) x 10^{-3}

Distribution of Lateral Forces Due to Eccentric Loading

E.Q.	Wall	$\frac{\Sigma H}{a}$	$\frac{Mc}{I}$	$\frac{\Sigma H}{a} \pm \frac{Mc}{I}$	Units of Rigidity RΣ (a)	H kips	Calculations
N-S	A	....	0.9	0.9	1.10	1.0	$\frac{\Sigma H}{a} = \frac{19.6}{1.19+0.83} = \frac{19.6}{2.02} = 9.7$ kips
	C	....	0.7	0.7	1.39	1.0	
	B	9.7	−0.7	9.0	1.19	10.7	$\frac{Mc}{I_R} = \frac{49.2c}{865} = .057$ c
	D	9.7	1.0	10.7	0.83	8.9	
E-W	A	7.9	0.6	8.5	1.10	9.4	$\frac{\Sigma H}{a} = \frac{19.6}{1.10+1.39} = \frac{19.6}{2.49} = 7.9$ kips
	C	7.9	−0.5	7.4	1.39	10.3	
	B	....	0.4	0.4	1.19	0.5	$\frac{Mc}{I_R} = \frac{31.4c}{865} = .036$ c
	D	....	0.6	0.6	0.83	0.5	

The "polar" moment of inertia is determined, using relative rigidities instead of areas. Then the distribution of the lateral force and the torsional moment to the walls is done with the use of the basic formula P/A ± Mc/I, except that in this case the denominators are still in terms of rigidities and not section. Finally, a correction is made which cancels out the rigidity unit and leaves the force H in kips (or pounds) which each wall is to resist. These forces are entered in Table 5-6 under the column "H". Values of v and F_b are then determined as described for Table 5-4.

Another method of determining the polar moment of inertia of the units of rigidity would be to take each pier rather than each wall; i.e., to consider each pier as an individual element in the torsional resistance. This may or may not be more work, depending upon the individual preference. With experience in this work, an engineer learns when torsional effects are sufficient to require tedious calculations. Unless the eccentricity is considerable, torsional motion may be only a small additional factor to the stresses caused by translational motion.

TABLE 5-9

DETERMINATION OF MAXIMUM AVERAGE VALUE OF F_a

			Weight lb.
Roof D.L.	1 ft. x 8 ft. x 15 lb. per sq. ft.	=	120
Floor D.L.	1 ft. x 8 ft. x 15 lb. per sq. ft.		120
Floor L.L.	1 ft. x 8 ft. x 40 lb. per sq. ft.	=	320
Partitions	1 ft. x 8 ft. x 10 lb. per sq. ft.	=	80
Wall	1 ft. x 18 ft. x 95 lb. per sq. ft. x 80 per cent	=	1,370
			2,010 lb.

$$F_a = \frac{2,010}{9 \times 12} = 18.6 \text{ psi}$$

Critical Stresses

Shear

Maximum actual shear = 8.8 psi in pier A-2
Maximum allowable shear = 50 x 1⅓ = 67 psi

Compression

Maximum actual compression = 50.7 psi in pier D-1
Maximum allowable axial compression = 300 x 1⅓ = 400 psi

Tension in pier

$$A_s = \frac{M}{f_s\, jd} = \frac{155 \times 10^3 \text{ (pier D-2)}}{1\tfrac{1}{3} \times 20 \times 10^3 \times \tfrac{7}{8} \times (96-2).} = .072 \text{ sq. in.}$$

No. 4 bars = 0.20 sq. in.

Table 5-10 shows some comparisons of blast with the earthquake forces assumed in the problem. As indicated, with proper development of roof and floor diaphragms, bonding, anchoring, tying and attention to details, this two-story reinforced masonry residence would meet recommendations for blast design as well as severe earthquake requirements.

TABLE 5-10

BLAST EFFECTS

On Walls Parallel to the Direction of Blast

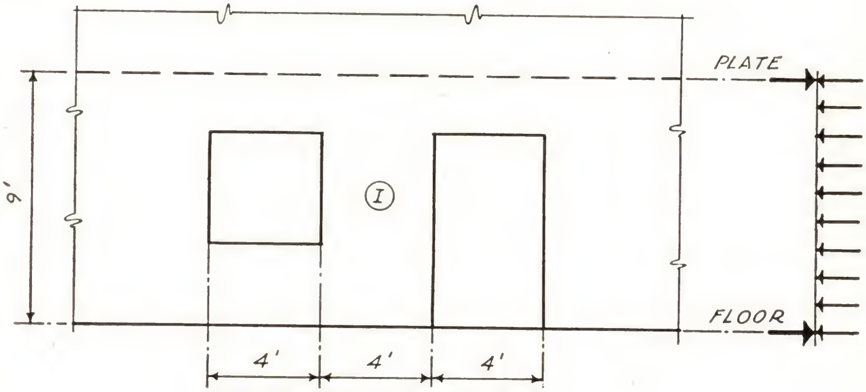
ΣH for blast = (1 + 19.5 ft.) x 28 ft. x 90 lb. per sq. ft. = 51,600 lb.

Blast stresses will be $\frac{51,600}{19,600} = 2.64$ times those due to 10.9 per cent E.Q.

Shear = 2.64 x 8.8 psi = 23.2 psi
Compression = 2.64 x 50.1 psi = 132 psi
Area Steel = 2.64 x .072 sq. in. = 0.19 sq. in.
The above values are all satisfactory.

Note: 1 ft. added to contributing height to allow for eave, roof pitch, drag, etc.

On Walls Normal to Direction of Blast (see diagram).



ELEVATION OF WALL "A"

Note: In an actual design, window areas which will offer no resistance to blast forces need not be included in the tributary area.

w = 90 lb. per sq. ft. x 8 = 720 lb. per ft.

L = 9 ft. (Use M = 1/12 Wl for effect of continuity).

W = 720 x 9 = 6,480 lb.

M = 1/12 x 6,480 x 9 x 12 = 58,320 in.-lb.

$$\text{Actual } f_m = \frac{M}{1/6\ bd_s^2} = \frac{58{,}320}{1/6 \times 48\ (4.5)^2} = \frac{58{,}320}{162} = 360 \text{ psi}$$

Allowable $f_m = 500 \times 1\frac{1}{3} = 667$ psi

Note that F_a is neglected because it is very small.

$$A_s = \frac{M}{f_s\ jd} = \frac{58{,}320}{20{,}000 \times 1\frac{1}{3} \times \frac{7}{8} \times 4.5} = 0.56 \text{ sq. in. required}$$

Use No. 4 bars at 24-in. centers vertically and No. 4 bars at each jamb.

Spandrel

The moment from the 2nd story shears taken by the spandrel is assumed to be ⅓ of that from the 1st story. Therefore, the maximum moment due to lateral forces to which the spandrel will be subjected = ½ x 155 x 2.64 x 1.33 = 272 in.-kips

$$\text{Actual } f_s = \frac{M}{A_s\ jd} = \frac{272{,}000}{0.20 \times \frac{7}{8} \times (60-2)} = 26{,}700 \text{ psi}$$

Allowable $f_s = 20{,}000 \times 1\frac{1}{3} = 26{,}700$ psi

$$\text{Actual } f_m = \frac{M}{1/6\ bd^2} = \frac{272{,}000}{1/6 \times 9 \times (60-2)^2} = 539 \text{ psi}$$

Allowable $f_m = 500 \times 1\frac{1}{3} = 667$ psi.

Stresses do not exceed allowable and steel requirement is less than actual.

508. ONE-STORY COMMERCIAL BUILDING

A one story commercial building with reinforced brick masonry walls and pilasters is illustrated in Fig. 5-6.

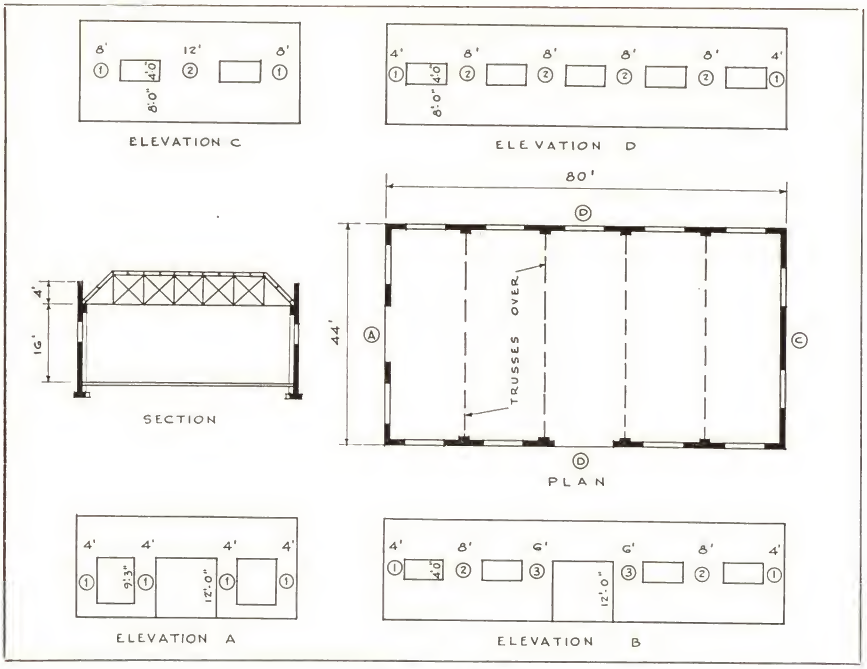


Fig. 5-6
One-Story Commercial Building

Trusses are assumed which are carried on reinforced masonry pilasters. The bays are 16 ft. wide and the span 44 ft. In this type of construction, the design could be done in assumed "box" action or in "bent" action. In the latter, each bay would have to develop lateral resistance for its tributary area. Sufficient moment value between the trusses and the pilasters, perhaps supplemented by some fixity at the base of the pilasters, would have to be provided. The method of this analysis is well illustrated in most texts on structural design and will not be repeated here. Since the roof plan ratio is 80/44 or only 1.82, it is possible and it will be assumed here that an adequately rigid and strong diaphragm can be provided.

The seismic analysis, which in this case also considers torsion due to the asymmetry of the walls, is shown in Tables 5-11, 5-12, 5-13 and 5-14. Note that a correction factor is applied for the thicker walls used in the front or south face. Heavier walls are desirable here because of the great amount of wall openings. The eccentricity is considerable for the earthquake in the direction parallel to walls A and C because wall A, even with thicker masonry, has much less rigidity than wall C. Note that the torsional motion adds more force to wall A than the translational earthquake motion. Such walls must be properly analyzed, designed, detailed, and constructed to provide satisfactory service under severe lateral forces. Actually, after determining the force to be applied to this wall, it should be calculated as a rigid frame according to conventional methods. Some engineers would design such an open wall for at least half the total seismic force.

TABLE 5-11

9-IN. RBM WALLS

1952 U.B.C. EARTHQUAKE FOR ZONE 3

Pier	No.	t in.	h in.	d in.	h/d	R_1*	$R\Sigma$	R	H lb.	ΣP lb.
A–1	4	14.25	111	48	2.32	82	328	1.00		11,700
							328	1.00	11,700	11,700
C–1	2	9	48	96	0.50	610	1,220	.56		16,900
–2	1	9	48	144	0.33	950	950	.44		13,300
							2,170	1.00	30,200	30,200
B–1	2	9	48	48	1.00	250	500	.19		3,900
–2	2	9	48	96	0.50	610	1,220	.47		9,600
–3	2	9	48	72	0.67	440	880	.34		7,000
							2,600	1.00	20,500	20,500
D–1	2	9	48	48	1.00	250	500	.17		3,600
–2	4	9	48	96	0.50	610	2,440	.83		17,800
							2,940	1.00	21,400	21,400

* R_1 of pier A-1 = 52 × 14.25/9 = 82

9-IN. RBM WALLS

1952 U.B.C. EARTHQUAKE FOR ZONE 3—Continued

Pier	P lb.	A sq. in.	v psi	M in.-lb. x 10^{-3}	S in.^{3} x 10^{-3}	F_b psi	F_c psi	F_a psi	ΣF psi
A–1	2,900	655	5.1	161	5.03	32.0	5.4	34.7	72.1
C–1	8,450	845	11.4	203	13.2	15.4	9.3	34.7	59.4
–2	13,300	1,280	11.9	319	30.3	10.5		34.7	54.5
B–1	1,950	414	5.4	47	3.18	14.8	2.9	34.7	52.4
–2	4,800	845	6.5	115	13.2	8.7	2.9	34.7	46.3
–3	3,500	630	6.3	84	7.35	11.4	2.9	34.7	49.0
D–1	1,800	414	5.0	43	3.18	13.5	3.0	34.7	51.2
–2	4,450	845	6.0	107	13.2	8.1	3.0	34.7	45.8

Critical Stresses

Shear

Maximum actual shear = 11.9 psi in pier C–2
Maximum allowable shear = 50 x 1⅓ = 67 psi

Compression

Maximum actual compressive stress = 72.1 psi in pier A–1
Maximum allowable axial compressive stress = 300 x 1⅓ = 400 psi

Tension in pier

$$A_s = \frac{M}{f_s\ jd} = \frac{161 \times 10^3 \text{ in.-lb. (pier A-1)}}{10^3 \times 20 \times 1\frac{1}{3} \times \frac{7}{8} \times (48-2)} = .15 \text{ sq. in.}$$

TABLE 5-12

9-IN. RBM WALLS

1952 U.B.C. EARTHQUAKE FOR ZONE 3

Determination of Lateral Force to be Applied

Earthquake

		x .133
Dead load down to mid height of wall		
Roof 44 ft. x 80 ft. x 15 lb. per sq. ft.	= 52,800 lb.	7,000 lb.
Walls 90 per cent x (44+80)2 x 95 lb. per sq. ft. x 12 ft.	= 262,000	34,800
	314,800 lb.	41,800 lb.

From 1952 U.B.C. (Zone 3), C = .133

Therefore, $\Sigma H = .133 \times 314{,}800$ lb. $= 41{,}800$ lb.

Determination of Height of Application of Latera Forces

7.0 kips	x 20 ft.	= 140 ft.-kips
34.8	x 15.5	= 540 ft.-kips
41.8 kips		680 ft.-kips

$$\text{Moment arm above floor} = \frac{680}{41.8} = 16.2 \text{ ft.}$$

At south wall, moment arm above sill = 16.2 − 2.75 = 13.45 ft.

At other walls, moment arm above sill = 16.2 − 8 = 8.2 ft.

Determination of Maximum Average Value of F_a

Note: For purposes of simplicity, the effect of the pilasters upon the values of v, F_b, and F_c will be neglected because it will be small. The value of F_a which will be used will be that equivalent to the case at the south wall.

Roof D.L.	= 14 ft. x 8 ft. x 15 lb. per sq. ft.	= 1,680 lb.
Wall	= 148 lb. per sq. ft. x 14 ft. x 8 ft.	= 16,580
Wall	= 148 lb. per sq. ft. x 4 ft. x 9.25 ft.	= 5,470
		23,730 lb.

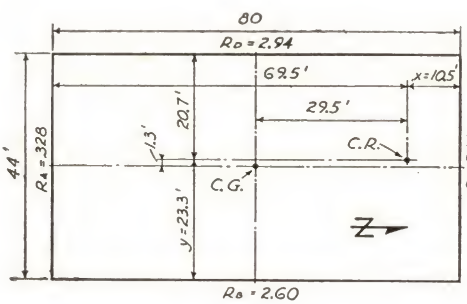
$$F_a = \frac{23{,}730 \text{ lb.}}{14.25 \times 48} = 34.7 \text{ psi}$$

TABLE 5-13

DISTRIBUTION OF LATERAL FORCES

$$\bar{y} = \frac{\Sigma Ry}{\Sigma R} = \frac{2.94 \times 44}{2.94 + 2.60} = 23.3 \text{ ft.}$$

$$\bar{x} = \frac{\Sigma Rx}{\Sigma R} = \frac{.328 \times 80}{.328 + 2.17} = 10.5 \text{ ft.}$$



$M_{N\text{-}S} = 41.8$ kips x 1.3 ft. = 54.3 ft.-kips.

$M_{E\text{-}W} = 41.8$ kips x 29.5 ft. = 1,230 ft.-kips.

Moment of Inertia of Walls About Point of Rotation (See Table 5-7)

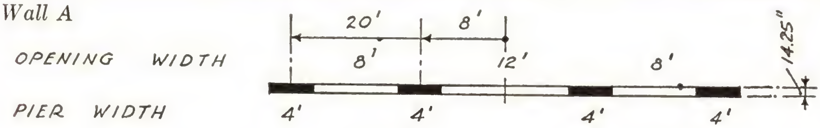
Wall	R_Σ (a)	c	c^2	ac^2
A	.328	69.5	4,830	1,580
C	2.17	10.5	110	238
B	2.60	23.3	545	1,420
D	2.94	20.7	428	1,250
				$I_R = 4,488$

Distribution of Lateral Forces Due to Eccentric Loading (See Table 5-7)

E.Q.	Wall	$\frac{\Sigma H}{a}$ kips	$\frac{Mc}{I}$ kips	$\frac{\Sigma H}{a} \pm \frac{Mc}{I}$	Units of Rigidity R (a)	H kips	Calculations
N-S	A	....	0.8	0.8	.328	0.3	$\frac{\Sigma H}{a} = \frac{41.8}{2.60+2.94} = \frac{41.8}{5.54} = 7.6$
	C	....	0.1	0.1	2.17	0.2	
	B	7.6	0.3	7.9	2.60	20.5	$\frac{Mc}{I} = \frac{54.3\,c}{4488} = .0121\,c$
	D	7.6	−0.3	7.3	2.94	21.4	
E-W	A	16.7	19.1	35.8	.328	11.7	$\frac{\Sigma H}{a} = \frac{41.8}{.33+2.17} = \frac{41.8}{2.50} = 16.7$
	C	16.7	−2.9	13.9	2.17	30.2	
	B	....	6.4	6.4	2.60	16.6	$\frac{Mc}{I_R} = \frac{1230\,c}{4488} = .275\,c$
	D	....	5.7	5.7	2.94	16.8	

TABLE 5-14
DETERMINATION OF OVERTURNING STRESS

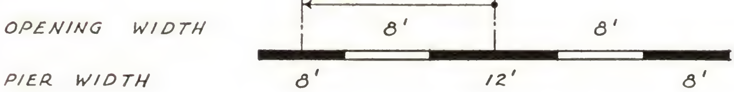
Wall A



$$I_{wall} = \frac{1}{12} \times 4 \times 4^3 \times \frac{14.25}{12} + 2 \times 4 \times (8^2 + 20^2) \times \frac{14.25}{12}$$

$$= 26 + 4,430 = 4,456 \text{ ft.}^4$$

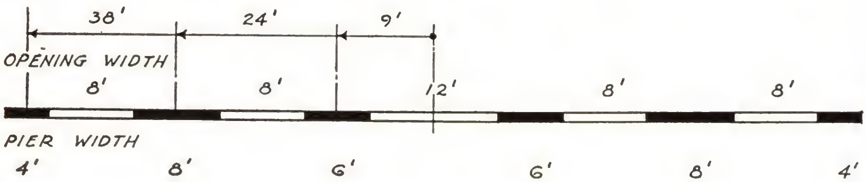
Wall C



$$I_{wall} = \frac{1}{12} \times 2 \times \frac{9}{12} \times 8^3 + \frac{1}{12} \times \frac{9}{12} \times 12^3 + 2 \times \frac{9}{12} \times 8 \times 18^2$$

$$= 64 + 108 + 3,900 = 4,072 \text{ ft.}^4$$

Wall B



$$I_{wall} = 2 \times \frac{1}{12} \times \frac{9}{12} \times (4^3 + 8^3 + 6^3) + 2 \times \frac{9}{12} \times 6 \times 9^2 + 2 \times \frac{9}{12}$$

$$\times 8 \times 24^2 + \frac{9}{12} \times 4 \times 38^2 \times 2$$

$$= 99 + 729 + 6{,}920 + 8{,}660 = 16{,}408 \text{ ft.}^4$$

Wall D

I_{wall} is practically the same as that of Wall B.

Maximum Stress The maximum overturning stress for each wall will be used for each pier for simplicity, since it is anticipated that compression will not be critical.

$$\textit{Wall A} = \frac{Mc}{I} = \frac{11{,}700 \text{ lb.} \times 13.45 \text{ ft.} \times 22 \text{ ft.}}{4456 \text{ ft.}^4} = 775 \text{ lb. per sq. ft.} = 5.4 \text{ psi}$$

$$\textit{Wall C} = \frac{Mc}{I} = \frac{30{,}200 \text{ lb.} \times 8.2 \text{ ft.} \times 22 \text{ ft.}}{4{,}072 \text{ ft.}^4} = 1{,}330 \text{ lb. per sq. ft.} = 9.3 \text{ psi}$$

$$\textit{Wall B} = \frac{Mc}{I} = \frac{20{,}500 \text{ lb.} \times 8.2 \text{ ft.} \times 40 \text{ ft.}}{16{,}408 \text{ ft.}^4} = 410 \text{ lb. per sq. ft.} = 2.9 \text{ psi}$$

$${}^*\textit{Wall D} = \frac{Mc}{I} = \frac{21{,}400 \text{ lb.} \times 8.2 \text{ ft.} \times 40 \text{ ft.}}{16{,}408 \text{ ft.}^4} = 428 \text{ lb. per sq. ft.} = 3.0 \text{ psi}$$

*Approximate.

The unit stresses due to the earthquake force are compared to blast stresses in Table 5-15. Note that the unit stresses are satisfactory for the 90 lb. per sq. ft. blast force. Also, the most narrow pier of the building (No. A-1), which has the most tributary area, is checked for the blast force normal to the wall surface. Although this pier is satisfactory for compressive stress, the vertical reinforcement must be substantial as indicated. Actually, this entire front wall should be illustrated on the drawings with adequate sections and elevations to show the builder how to place and where to lap all bars. Needless to say, the reinforcement must be located so as to develop full continuity. All wall sections should be checked for normal forces—the narrow pier (A-1) may not be the most critical in this structure because of its thickness and double curtain of reinforcement.

On Walls Parallel to the Direction of Blast

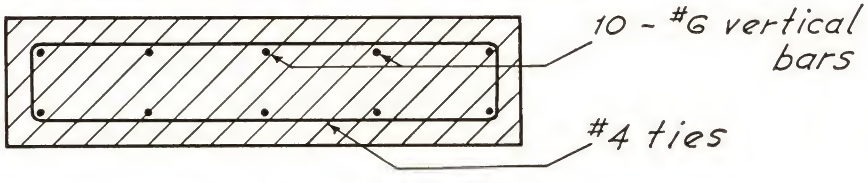
ΣH for blast = 15 ft. x 80 ft. x 90 lb. per sq. ft. = 108,000 lb.

Blast stresses will be $\frac{108{,}000}{41{,}800} = 2.58$ times that due to a 13.3 per cent E.Q. when forces in an east-west direction are considered.

Blast stresses will be $\frac{15 \times 44 \times 90}{41{,}800} = \frac{59{,}400}{41{,}800} = 1.42$ times that due to a 13.3 per cent E.Q. when forces in a north-south direction are considered.

Maximum shear due to blast = 2.58 x 11.9 (pier C–2) = 30.7 psi
Maximum compressive stress due to blast = 72.1 x 2.58 (pier A–1) = 186 psi
Area of steel = 0.15 sq. in. (pier A–1) x 2.58 = 0.39 sq. in.

The above stresses are all within the allowable stresses and the steel area of 0.39 sq. in. is less than the 0.40 sq. in. (two No. 4) furnished.



Plan of pier

On Walls Normal to the Direction of Blast

Consider pier A–1 adjacent to door jamb which will take a tributary loading area of 14 ft. wide x 16 ft. high = 224 sq. ft.

$M = \frac{1}{8} \times 224$ sq. ft. x 90 lb. per sq. ft. x (16 x 12) in. = 485,000 in.-lb.

$$\text{Actual } f_m = \frac{M}{1/6\ bd_s^2}\text{(approximate)} = \frac{485{,}000\ \text{in.-lb.}}{1/6 \times 48\ \text{in.} \times (10\ \text{in.})^2} = 606\ \text{psi}$$

Allowable $f_m = 500 \times 1\frac{1}{3} = 667$ psi

(Note that F_a is neglected because of its small value.)

$$A_s = \frac{M}{f_s\ jd} = \frac{485{,}000}{20{,}000 \times 1\frac{1}{3} \times \frac{7}{8} \times 10} = 2.1 \text{ sq. in., or five No. 6 bars}$$

(5 x .44 = 2.20 sq. in.)

509. SEISMIC FORCES

The examples in this chapter have been analyzed with the earthquake factors of the Uniform Building Code, Zone 3, instead of with the lower factors recommended in Chapter 4. This is not an inconsistency—the recommendations still apply. The reasons for using the greater coefficients are twofold:

(1) To indicate that even the highest factors in force for low structures can be met with reinforced brick masonry construction, often with considerable reserve strength.

(2) To indicate to the reader that the actual seismic factor used is of less importance for such low rigid structures than the integrity and experience of the designer, his basic assumptions, judgment and attention to detail.

The analysis is an important aid to design judgment, particularly for those of little experience in seismic motion and damage. However, calculations are not the required end result as is so often assumed—the important thing is to obtain a *building* which is lateral-force resistant, not merely a set of calculations. To do this, the layout, the analysis, judgment, the design, the drawings, the construction and the inspection must all be continuously good. If they are, reinforced brick masonry can resist wind, earthquake and/or blast.

Perhaps no factor is so important as tying the various elements of the structure so that they can function together and remain together. This has been discussed in Section 408. Some examples will be given in the following chapter.

CHAPTER 6

REINFORCED BRICK MASONRY WALL SECTIONS AND DETAILS

601. INTRODUCTION

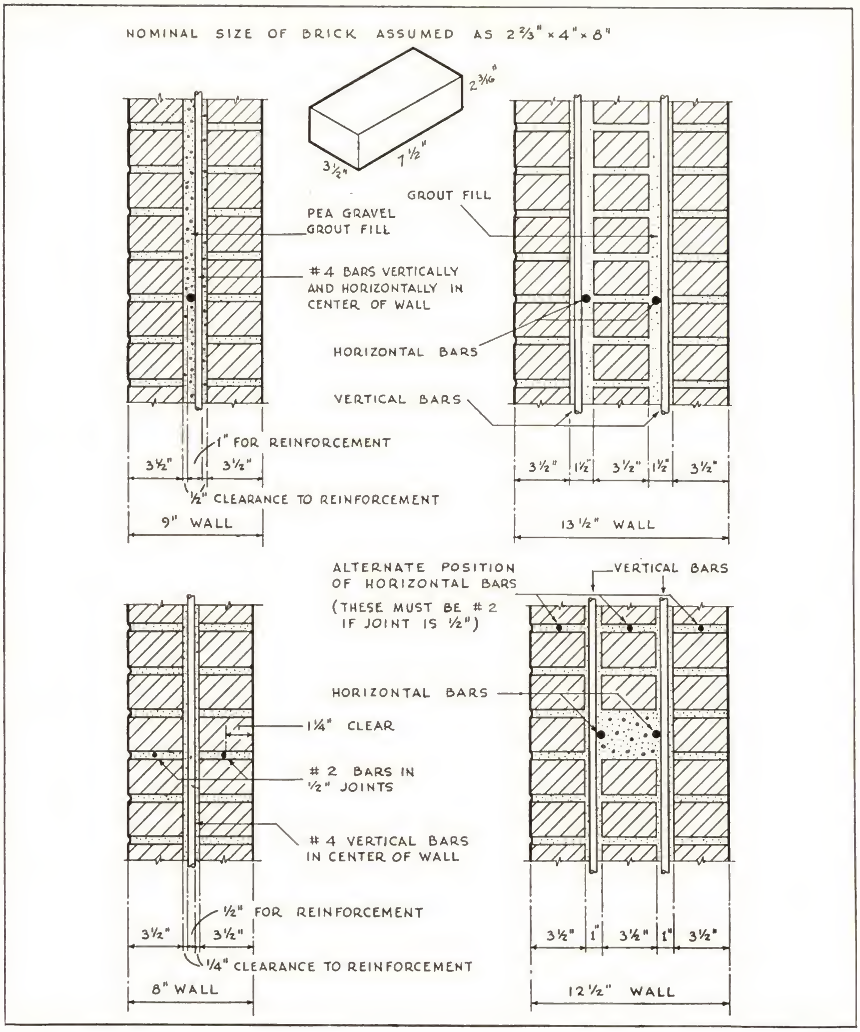
The various examples of wall sections, connections and details shown in this chapter are not standards to be used without question but merely suggested methods. Actually, the details to be used for any structure should depend upon the particular conditions. The designer must use ingenuity and, above all, keep in mind the basic factor of tying the parts of structures together, not only for the transfer of calculated stresses but for the intangible demands of chaotic and dynamic loading.

602. TYPES OF WALLS

Masonry walls may be bearing walls without pilasters, bearing walls with pilasters, curtain or filler walls, shear walls, or partition walls. Combinations often occur. An important thing in lateral force design is to consider all walls as part of the structure and to make them such. Even partitions have structural action in earthquakes since their mass creates inertia forces. These forces must be transmitted to points of lateral support. Whether this can be done without the partition functioning as a shear element of the building is a matter for the designer to decide and to so detail and control. Items left to chance or to those not familiar with the action of lateral forces will usually be wrong and fail in emergencies.

In Chapter 5 the building examples were of bearing walls, with and without pilasters. In general, it is advisable to use pilasters where wall heights are great or where concentrated loads occur. Bond beams, and occasionally intermediate wall beams, may also be used.

Curtain walls of multi-story buildings are used in conjunction with a framework of structural steel or reinforced concrete. Even though the walls are assumed to carry no load except their own weight, it is obvious that such walls will tend to deflect and be stressed as the framework yields under lateral forces. The relative rigidity of masonry walls with nominal openings is usually much greater than that of the building framework. Thus the walls will be subjected to a large part of the lateral force. Whether or not the filler walls of multi-story buildings should be designed to withstand all possible force without even local shear cracking (and energy absorption) is debatable. One thing is certain however: the walls should be tied to the framework at all points of contact and should be reinforced not only for strength but to provide valuable ductile properties and to cause the walls to "hold together" in the event of very severe disturbances. It is also desirable that the framework in slender, multi-story buildings has some lateral force resistance in itself, preferably in rigid frame (moment) action. The design recommendations of Chapter 4 have taken these factors into consideration.



Note: Wall thicknesses may be different where brick of different widths are used. Maintain ¼ in. clearance (minimum) between bars and masonry units, except No. 2 bars may be used in ½ in. joints.

Fig. 6-1
Typical Wall Sections

603. WALL SECTIONS

Fig. 6-1 illustrates various reinforced brick wall sections and thicknesses made with brick of the size shown. Variations are possible as brick sizes vary locally. In addition, some manufacturers produce special shaped brick with keys or lugs and also special units to provide for horizontal reinforcement. Needless to say, all spaces must be completely filled with mortar or grout for the wall section to function as intended under lateral forces.

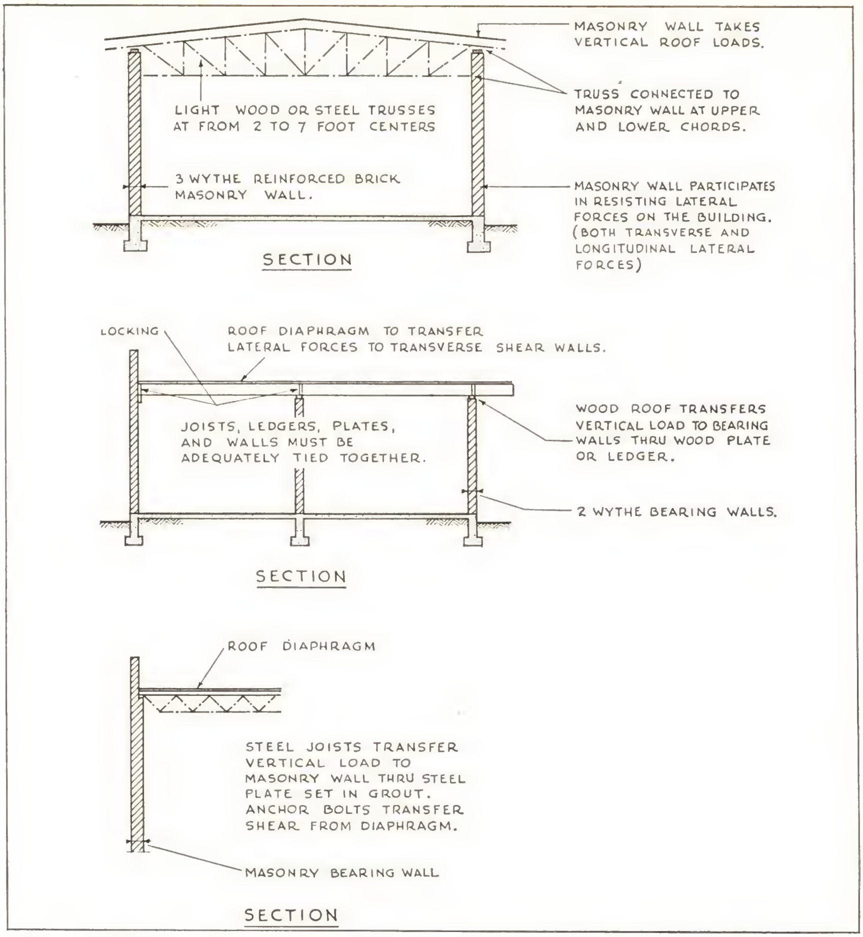


Fig. 6-2
Reinforced Brick Masonry Bearing Walls

Figs. 6-2, 6-3 and 6-4 indicate various constructions with brick curtain walls and bearing walls. A hypothetical problem will be analyzed using the steel frame of Fig. 6-4. Calculations are shown in Table 6-1 for earthquake and wind normal to the wall. The wind governs the design in this case.

TABLE 6-1

EARTHQUAKE AND WIND FORCES NORMAL TO WALL

Earthquake on Wall

The earthquake force to be applied normal to the wall is 20 per cent of the dead load of the wall. This percentage is required by the recommended code of Chapter 4, and by the 1952 Uniform Building Code.

Therefore: Lateral force = 0.20 x 95 lb. per sq. ft. = 19.0 lb. per sq. ft.

Wind on Wall

Wind force at 20 lb. per sq. ft. will govern the exterior wall. Assume the wall spans vertically 12 ft. and each 4-ft. vertical band between the windows resists wind on a 4-ft. + 2.67-ft. = 6.67-ft. wide band.

$M = 1/10\ wl^2$ (for partial continuity)

$= 1/10 \times (6.67 \text{ ft.} \times 20 \text{ lb. per sq. ft.}) \times (12 \text{ ft.})^2 = 1{,}920 \text{ ft.-lb.} = 23{,}000 \text{ in.-lb.}$

$S = 1/6\ bd^2 = 1/6 \times 48 \times (4.5)^2 = 162 \text{ in.}^3$ (approx.)

$$f_m \text{ (approximate)} = \frac{M}{S} = \frac{23{,}000}{162} = 142 \text{ psi which is below the allowable of}$$

$500 \times 1\tfrac{1}{3} = 667$ psi

$$A_s = \frac{M}{f_s j d_s} = \frac{23{,}000}{1\tfrac{1}{3} \times 20{,}000 \times \tfrac{7}{8} \times 4.5} = 0.22 \text{ sq. in.}$$

The No. 4 vertical bars at each jamb (two jamb bars per pier) would furnish the necessary reinforcement for wind.

Minimum Reinforcement for a 9-in. RBM Wall

Minimum reinforcement = 0.002 x 9 in. x 12 in. = .216 sq. in. per ft.

Vertically: No. 4 bars at $\frac{6.67}{4}$ ft. centers = 20 in. centers avg.
= 0.12 sq. in. per ft. width

Horizontally: No. 4 bars at 2-ft. centers. = $\underline{0.10}$
0.22 sq. in. per ft. width

(See example No. 2 on Fig. 6-5)

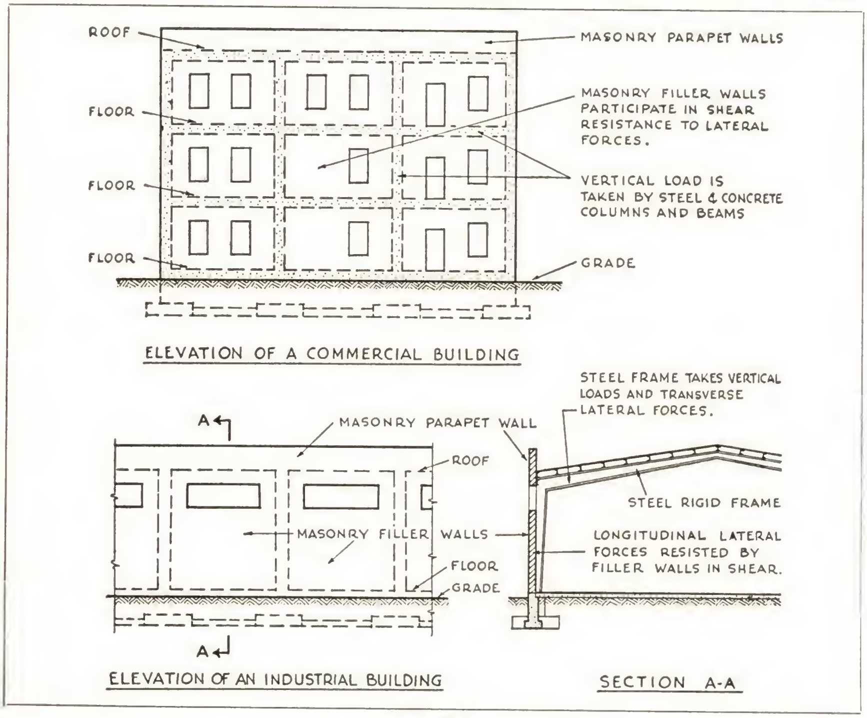


Fig. 6-3
Reinforced Brick Masonry Curtain Walls

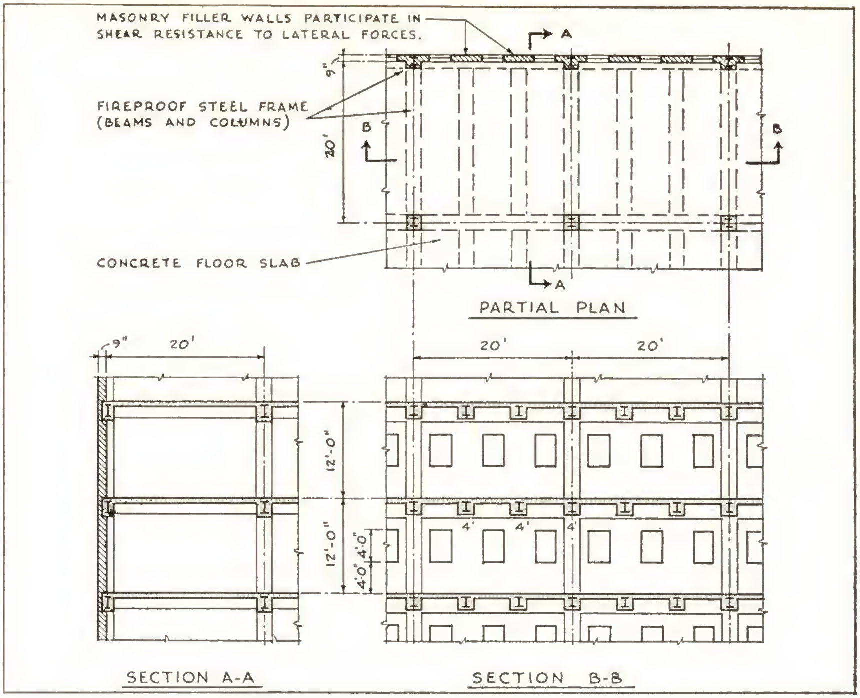


Fig. 6-4
One Bay of Skeleton Frame Construction

Using the requirement for full reinforcement that the minimum total area of steel must be 0.002 times the cross-sectional area of the wall, the reinforcement becomes No. 4 bars (½-in. round) placed vertically at 20-in. centers, and No. 4 horizontal bars at 24-in. centers to complete the area requirement. Fig. 6-5 offers a convenient method of selecting wall reinforcement.

Fig. 6-6 indicates the wall construction and the placing of bars. The important connection details are shown on subsequent sketches.

The same wall is checked for 90-lb. per sq. ft. blast pressure in Table 6-2. The vertical reinforcement must be increased for this assumed loading.

TABLE 6-2

BLAST ON WALL

Blast force = 90 lb. per sq. ft.

$M = 1/10\ wl^2$ (for partial continuity)

$= 1/10 \times (6.67 \times 90) \times 12^2 = 8{,}640$ ft.-lb. $= 104{,}000$ in.-lb.

f_m (approximate) $= \frac{M}{S_1} = \frac{104{,}000}{162} = 642$ psi which is below the allowable of $500 \times 1\frac{1}{3} = 667$ psi

$$A_s = \frac{M}{f_s j d_s} = \frac{104{,}000}{20 \times 10^3 \times 1\frac{1}{3} \times \frac{7}{8} \times 4.5} = 0.99 \text{ sq. in.}$$

Between the windows there would have to be placed five No. 4 vertical bars (at 11-in. centers).

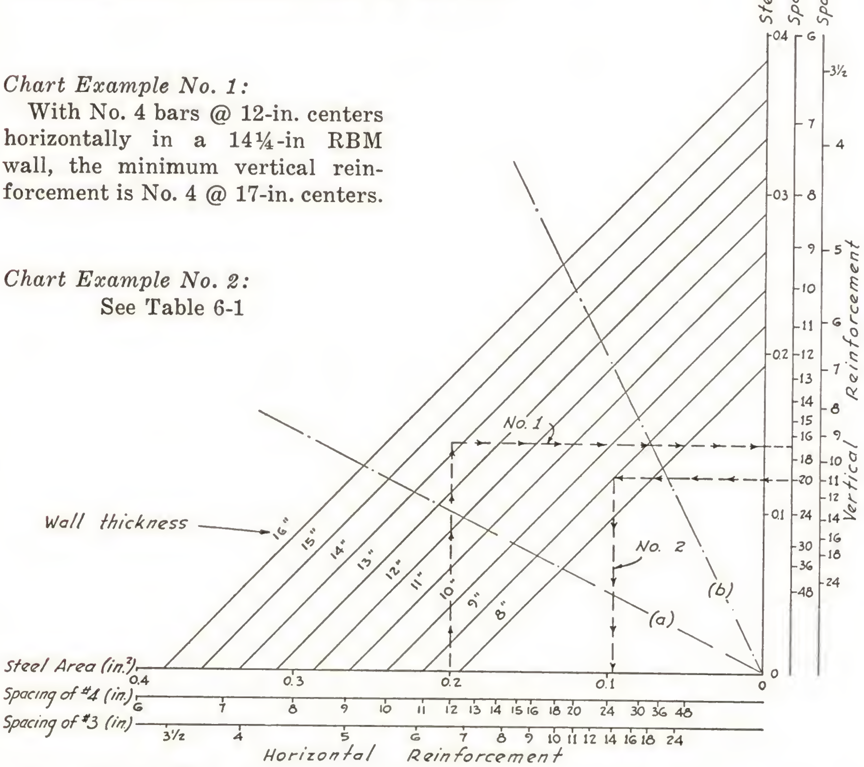
Thus, the wall reinforcement would be similar to that shown on Fig. 6-6, except that the vertical reinforcement would be No. 4 at 11-in. centers or 0.22 sq. in. per ft. of width.

The minimum area required of horizontal reinforcement would then be $\frac{1}{3} \times 0.002 \times 9 \times 12 = 0.072$ sq. in., or No. 4 bars at 32-in. centers maximum. (Refer to Note on Fig. 6-5.)

Chart Explanation: Select a point on the line which corresponds to the wall thickness being used and read the vertical and horizontal reinforcement directly opposite. This point must lie between lines (a) and (b).

Chart Example No. 1:
With No. 4 bars @ 12-in. centers horizontally in a 14¼-in RBM wall, the minimum vertical reinforcement is No. 4 @ 17-in. centers.

Chart Example No. 2:
See Table 6-1



Note: Basis of Chart: Total area of steel not less than 0.002 times the cross-sectional area of the wall, not more than 2/3 of which may be used in either direction. (See Section 308(h)5)

Fig. 6-5

Chart Giving Minimum Reinforcement for RBM Walls

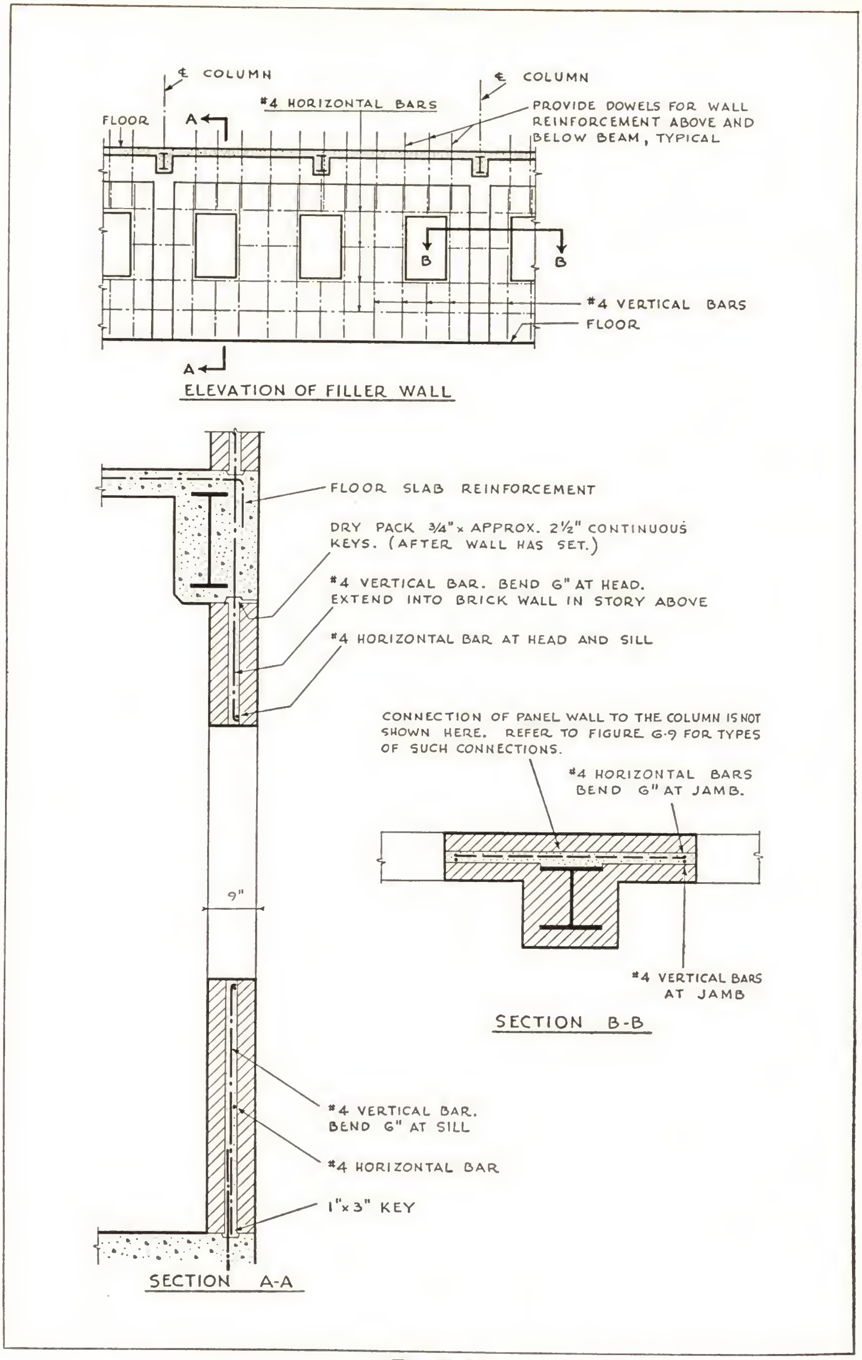


Fig. 6-6
Wall Construction and Placement of Bars

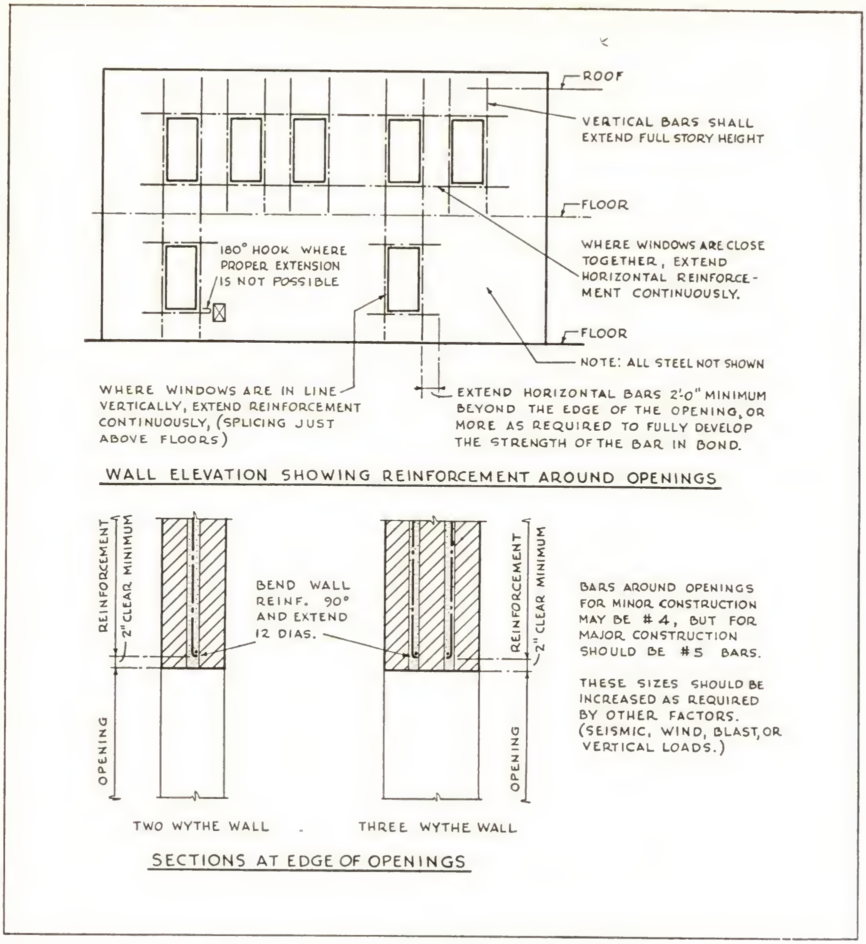


Fig. 6-7
Reinforcement Around Openings

Figs. 6-7 and 6-8 indicate additional details and considerations in placing wall reinforcement.

604. CONNECTIONS AND ATTACHMENTS

There are many possible ways of connecting walls to frames, floors and roofs. Several are illustrated by Figs. 6-9 and 6-10 in order to suggest methods to the reader. The important points are (1) to make the connections as often as possible to avoid large stress concentrations, (2) to make the connections adequate for the assumed forces, shears, etc., and (3) to make them as simple and as easy to build as possible, consistent with the necessary results. The methods, sizes, and details shown should be altered as required for any particular problem. Most of the details indicated, such as the anchorages of beams into walls, and others, are effective only if continuous bond beams are provided.

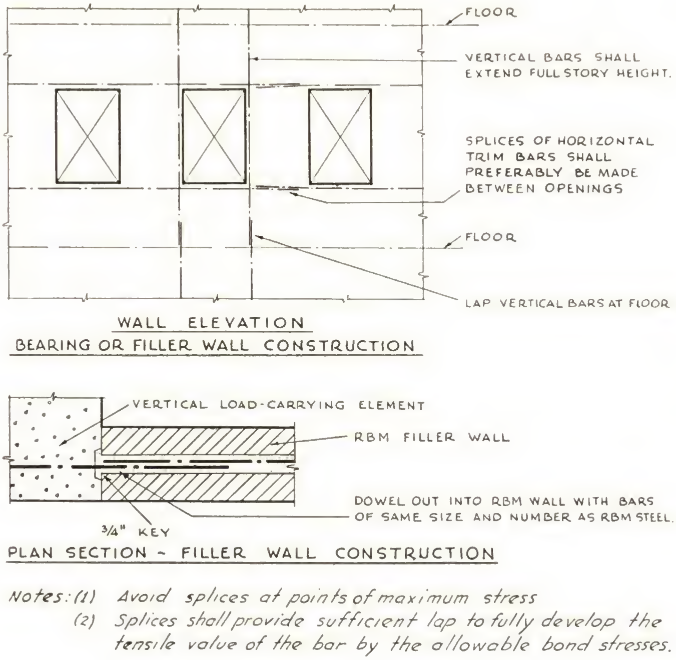


Fig. 6-8
Lap Splices and Dowels

605. BOND BEAMS

A bond beam is not only or necessarily a beam—it is a "jack of all trades." It may be considered as an element which is placed continuously around all walls at each floor level and at the roof level, and which has strength and rigidity in tension, compression, flexure and shear. It also helps in providing anchorage for bolts, straps, bars, etc. Often the frame (if there is one) can act as the bond beam. Where there is no frame, reinforcing bars can be provided as shown in Fig. 6-11 or in other ways to produce the desired result. The bond beam may function as a member of a horizontal bracing system (truss) or as the flange of a plate girder (the diaphragm). It will often be loaded horizontally and sometimes vertically as a beam and must be designed accordingly. All reinforcement must be placed continuously and around all corners.

Many manufacturers provide special shapes for forming bond beams.

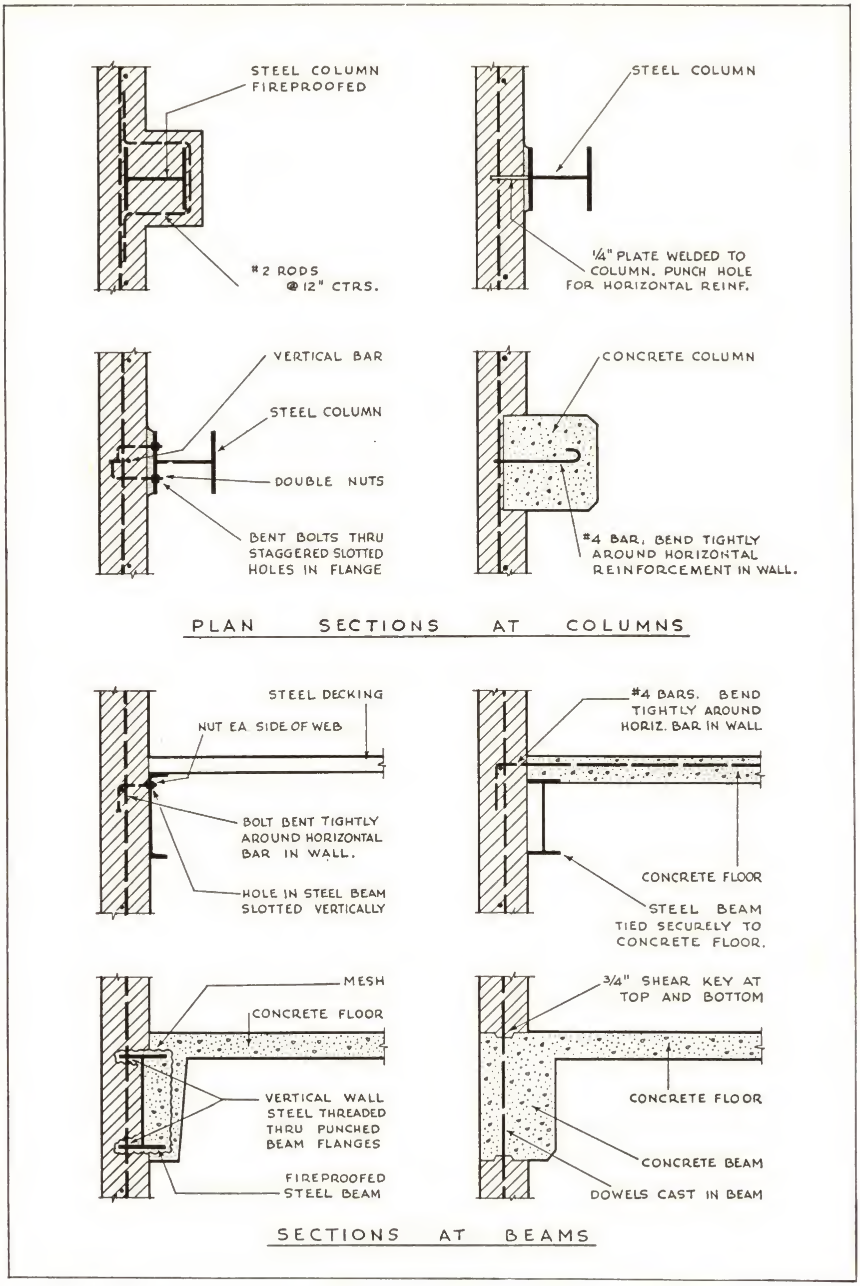


Fig. 6-9
Attachment of Panel Walls to Skeleton Frame

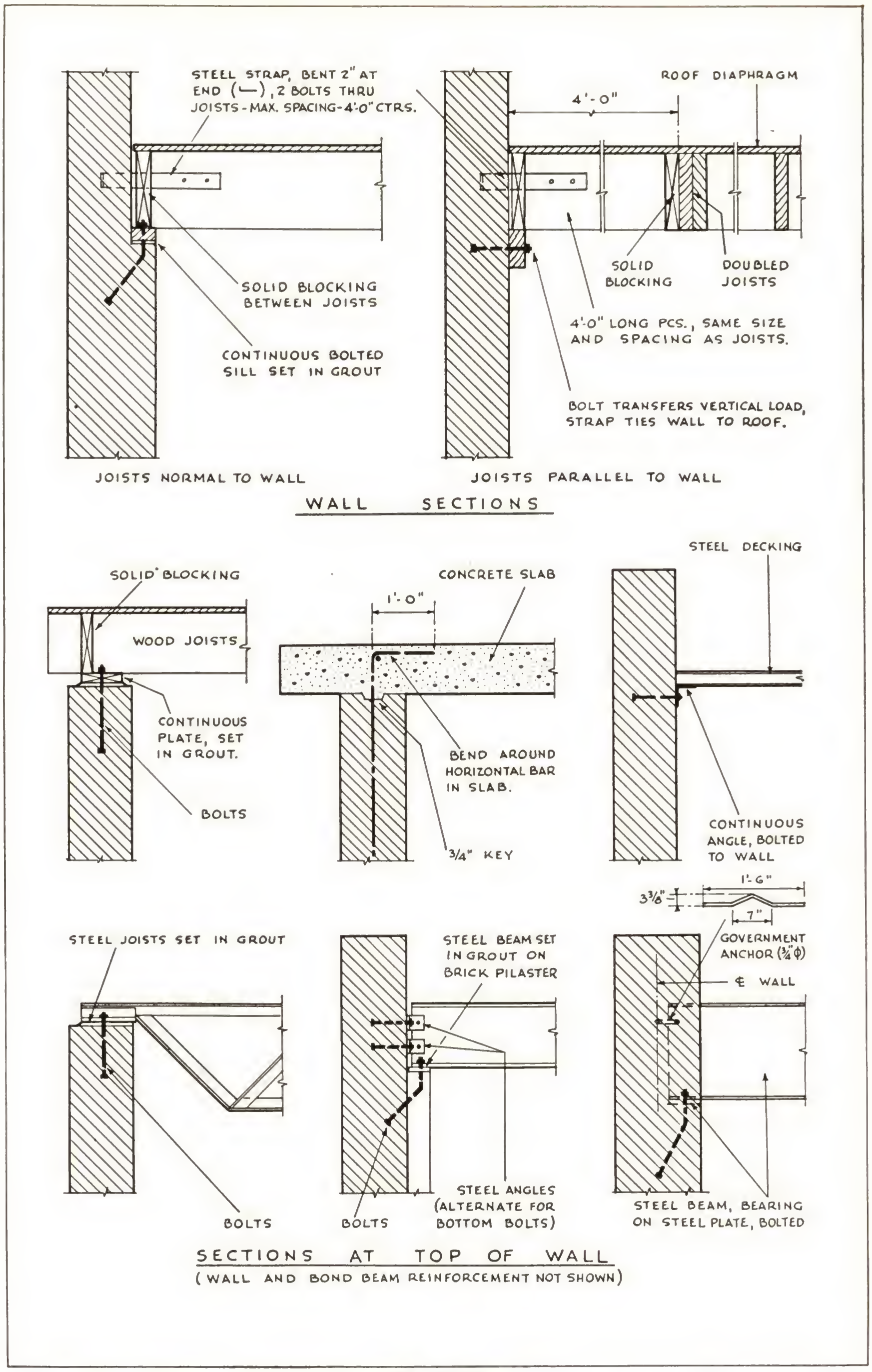


Fig. 6-10
Attachment of Floors and Roofs to Walls

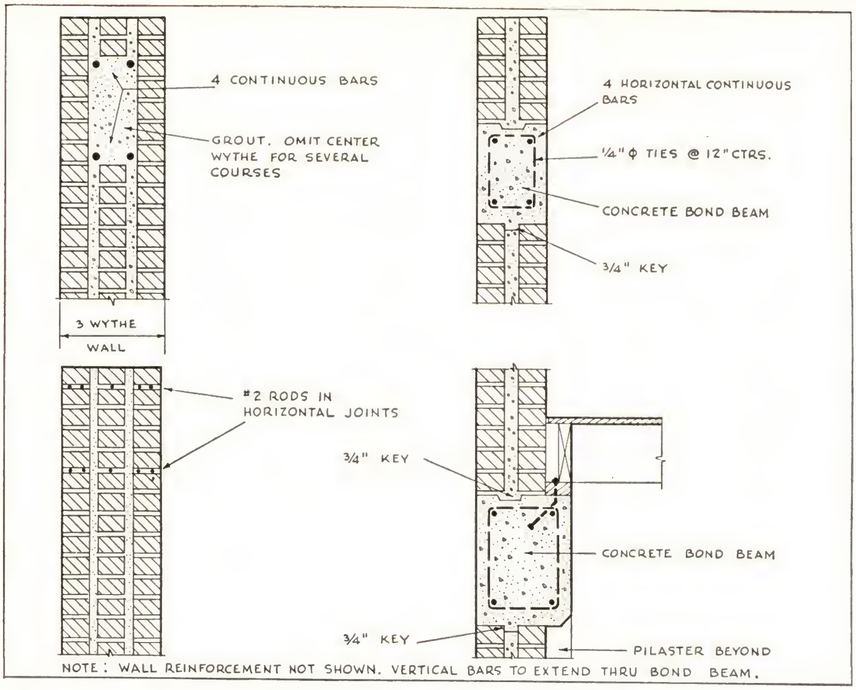


Fig. 6-11
Sections Through Bond Beams

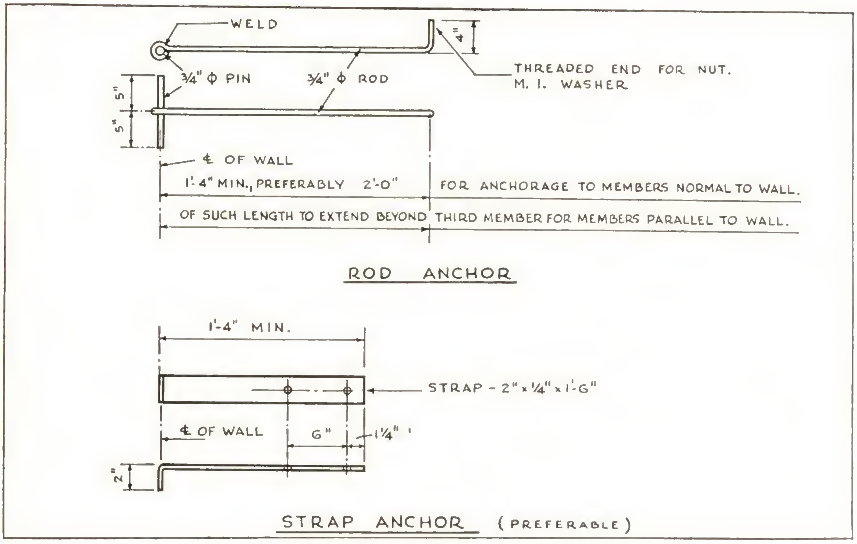


Fig. 6-12
Joist Anchors

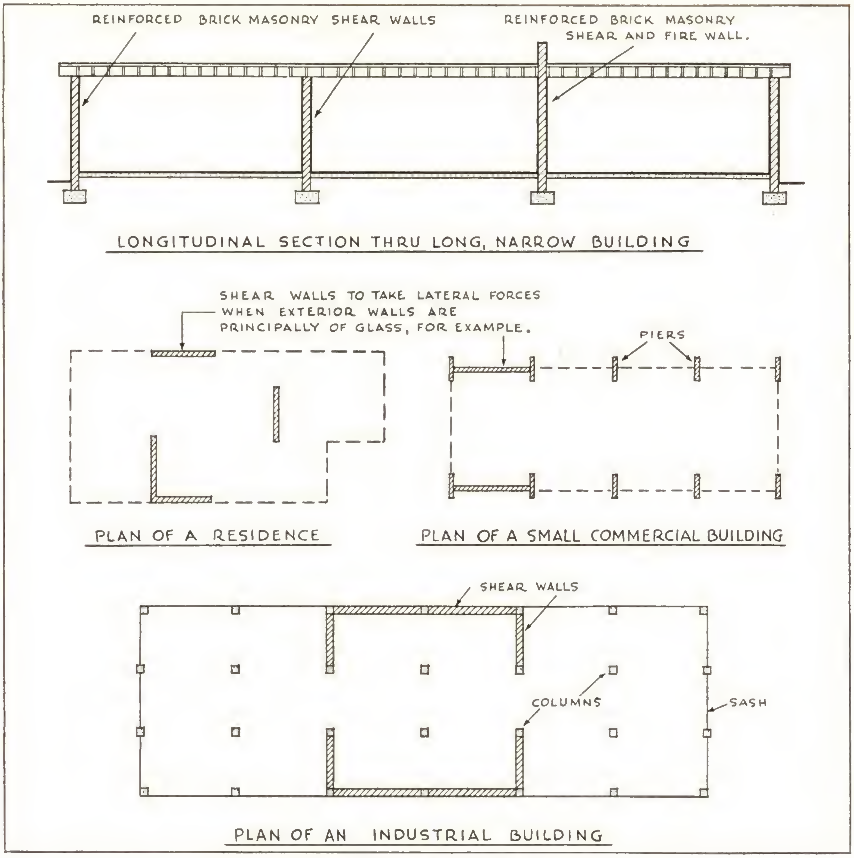


Fig. 6-13
Reinforced Brick Masonry Shear Walls

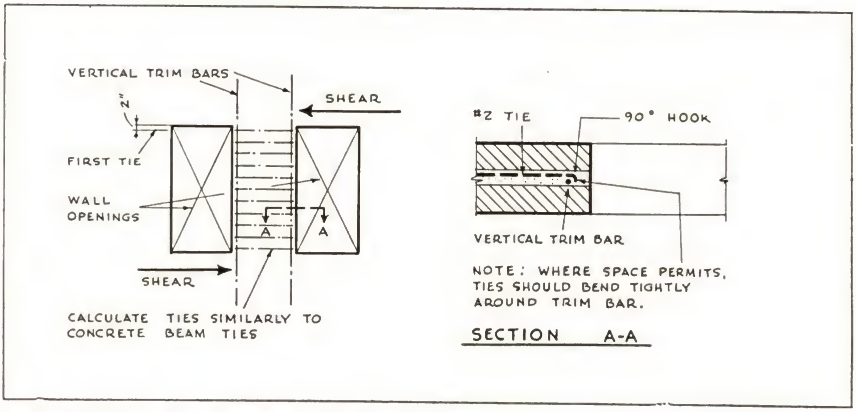
606. JOIST ANCHORS

All floors and roofs must be well tied and anchored into the walls. In wood construction this is sometimes difficult but nonetheless very important. The conventional rod anchor (see Fig. 6-12) is not as desirable as the strap anchor which has more positive action. Whatever the detail, anchors should be placed close together so as to avoid excessive stress concentrations. They must be conscientiously designed, detailed, placed and inspected.

607. SHEAR WALLS

It is often necessary or desirable to place reinforced masonry walls in the interior of a building to function in resisting shear from the lateral forces, and in providing rigidity and homogeneity of structural walls. In some cases, the architectural treatment is such as to make interior shear walls or moment-resisting frames a necessity. Great care must be taken to properly deliver and receive lateral forces to and from shear walls and also to make such walls an integral part of the structural system. Fig. 6-13 indicates some shear wall functions.

All that has been written regarding relative rigidity, diaphragms, bond beams, ties, anchors, etc., applies as well to unusual plan layouts where special walls are required. The development of a proper system becomes more difficult as the wall area of a building is reduced to create special architectural treatments. Occasionally it becomes necessary to provide shear reinforcement in a pier due to overstress of the masonry. Ties can be used (as for concrete beams) to reinforce special sections as shown in Fig. 6-14.



Note: Only the pier reinforcement is indicated.

Fig. 6-14

Steel Ties for Shear Reinforcement

In general, such critical areas should be avoided by the use of thicker walls or less wall openings. Too much steel is costly and its necessity in design usually indicates inadequate masonry area and/or a plan layout inherently poor in resistance to lateral forces.

CHAPTER 7

SPECIFICATIONS FOR MASONRY CONSTRUCTION

701. INTRODUCTION AND SCOPE

The following specifications are suggested as the masonry section covering structural clay products of a general construction specification and, when supplemented by the general conditions, plans, details and schedules, should provide the contractor with sufficient information to estimate costs and to construct the work. These specifications do not include concrete masonry units, glass block, stone or plain (unreinforced) concrete. Where these materials are included in the masonry contract, the specifications should be supplemented with suitable provisions. It will be found, however, that many of the requirements relating to brick and tile are equally applicable to other types of masonry construction.

The scope of the work covered by the masonry section of the specifications will vary for different jobs and should be indicated in the job specifications. In general, it will include:

(a) Furnishing all labor, materials, services, equipment and appliances required to complete the masonry work, including placing and building in reinforcement as indicated on the contract drawings and in the specifications.

Note: Steel reinforcement for masonry may be furnished either by the masonry subcontractor or the general contractor. If masonry reinforcement is to be furnished by the masonry subcontractor, it should be specifically stated in the scope.

(b) Building into masonry all bolts, anchors, nailing blocks, inserts, etc., necessary to construct the work, as furnished and located by the general contractors or others.

(c) Building in all door and window frames, vents, conduits, etc., as furnished and set by the general contractor or others.

(d) Furnishing all test specimens required by the contract drawings and specifications; such as specimens of materials, mortar cubes, masonry prisms and specimens cut from finished masonry.

(e) Cleaning masonry and removal of surplus material and waste.

(f) Completion of all masonry work in accordance with contract specifications and drawings.

The following are not, as a rule, included in the scope of the masonry work:

(g) Concrete work of any description, including furnishings and placing of anchorages and steel dowels in concrete.

(h) Furnishing and locating bolts, anchors, metal attachments, etc.

(i) Furnishing or placing any structural steel.

(j) Furnishing or placing any shoring, templates or similar carpentry work required by the specifications.

Copies of ASTM specifications referred to are available from the offices of the American Society for Testing Materials, 1916 Race Street, Philadelphia

3, Pennsylvania. Specifications of the Architectural Terra Cotta Institute and the Facing Tile Institute may be obtained from these institutes at 1520 18th Street, N. W., Washington 6, D. C.

This specification is in three parts. Section 702 covers general requirements applicable to both reinforced and unreinforced masonry. Section 703 covers workmanship, placing of reinforcement and other items in the construction of reinforced grouted brick masonry and bonded reinforced brick masonry. Section 704 covers unreinforced grouted brick and tile construction and includes workmanship, bonding requirements, etc.

The specification writer obviously should include in the job specifications only those sections which are applicable to the particular construction. He may change section titles as necessary.

702. GENERAL REQUIREMENTS

(a) Masonry Units.

1. The size of masonry units shall be ——— or as indicated on the plans.

2. Facing Brick shall conform to the requirements of ASTM Specifications C216-, Grade MW and Type ——— (specify Type FBX, FBS or FBA). The color and texture shall be similar to the sample approved by the architect.

3. Building Brick shall conform to the requirements of ASTM Specifications C62-, Grade SW for foundations or other structures below grade or in contact with earth, Grade MW for other exterior walls and Grade NW where not subject to the action of weather or soil.

4. Glazed Structural Facing Tile shall conform to the requirements of the specifications of the Facing Tile Institute for ——— (specify Color Ceramic Glaze, Clear Glaze or Salt Glaze, and grade). Color shall be ——— (see Facing Tile Institute color list).

5. Structural Clay Facing Tile shall conform to the requirements of ASTM Specifications C212-, Type ——— (specify Type FTX or FTS) and Class ——— (specify Standard or Special Duty).

6. Structural Clay Load-Bearing Wall Tile shall conform to the requirements of ASTM Specifications C34-, Grade LBX when exposed to weather or soil and Grade LB when not subject to the action of weather or soil.

7. Structural Clay Non-Load-Bearing Tile shall conform to the requirements of ASTM Specifications C56-.

8. Architectural Terra Cotta and Ceramic Veneer and the erection of same shall conform to the Standard Specifications for Furnishing and Erecting Architectural Terra Cotta and Ceramic Veneer of the Architectural Terra Cotta Institute.

(b) Mortar and Grout.

1. Mortar and mortar materials shall conform to the requirements of the property specifications of ASTM Specifications for Mortar for Unit Masonry, C270-, or shall consist of a mixture of cementitious materials and aggregate conforming to the requirements of Section 2 and the measurement and mixing requirements of Section 3 of Specifications C270-, and shall be proportioned within the limits given in Table 7-1 for each mortar type specified.

TABLE 7-1

MORTAR PROPORTIONS BY VOLUME

Mortar Type	Parts by Volume of Cement	Parts by Volume of ASTM C901–Masonry Cement	Parts by Volume of Hydrated Lime or Lime Putty	Aggregate, measured in a damp, loose condition
A–1	1	1 (Type II)	....	Not less than 2¼ and not more than 3 times the sum of the volumes of the cement and lime used.
	1	....	¼	
A–2	½	1 (Type II)	....	
	1	....	over ¼ to ½	
B	....	1 (Type II)	....	
	1	....	over ½ to 1¼	
C	....	1 (Type I or II)	....	
	1	....	over 1¼ to 2½	

2. Types A-1, A-2 and B grout shall be mortars as specified in Section 702(b)1 of the same types, respectively, to which water is added to produce consistency for pouring without segregation of the constituents of the mortar. After adding water to the mortar, all grout shall be stirred or worked at frequent intervals.

3. Pea gravel grout shall be Type A-1 grout to which pea gravel may be added equal in volume to not more than twice the volume of cement. Such pea gravel shall conform to the grading limits of Table 7-2.

TABLE 7-2

GRADING LIMITS FOR PEA GRAVEL IN GROUT

Sieve Size	Percentage (by weight) Passing Through Sieve
⅜	95—100
No. 3	45— 75
No. 4	10— 20
No. 8	0— 5

4. The consistency of mortar shall be adjusted to the satisfaction of the mason, but as much water shall be added as is compatible with convenience in using the mortar. If the mortar begins to stiffen from evaporation or absorption of a part of the mixing water, the mortar shall be retempered by adding water and remixing. Mortar and grout shall not be used after it has begun its set.

(c) Reinforcement. Metal reinforcement shall be plain or deformed steel bars, cold drawn steel wire, or structural steel sections as required by the drawings and specifications. Deformation of deformed bars shall conform to ASTM Standard Specifications for Minimum Requirements for Deformed Bars for Concrete Reinforcement, A305-.

Reinforcement shall conform in quality to standard specifications of the ASTM of the following applicable titles:

Billet-Steel Bars for Concrete Reinforcement, A15-
Rail-Steel Bars for Concrete Reinforcement, A16-
Axle-Steel Bars for Concrete Reinforcement, A160-
Cold-Drawn Steel Wire for Concrete Reinforcement, A82-
Welded Steel Wire Fabric for Concrete Reinforcement, A185-
Steel for Bridges and Buildings, A7-

(d) Storage of Materials.

1. All mortar materials shall be stored under cover in a dry place.

2. Brick and structural tile shall be piled on plank platforms in a dry location. During freezing weather all masonry units shall be protected with tarpaulins or other suitable material. Architectural terra cotta and structural facing tile, both glazed and unglazed, shall be covered at all times.

At the time of laying, all masonry units shall be clean and free from ice or other coatings.

3. Reinforcement shall be protected from the elements and before being placed shall be free from loose rust and other coatings, including ice, that will destroy or reduce the bond.

(e) Wetting of Brick and Tile.

1. All brick having absorption rates (determined in accordance with ASTM Specifications C67-) in excess of ¾ oz. per min. (.025 oz. per sq. in. per min.) shall be wetted sufficiently so that the rate of absorption when laid does not exceed this amount.

2. Structural tile having absorptions (1-hr. boil) of 12 per cent or more shall also be wetted before laying.

3. The units shall be wetted from 3 to 4 hr. before they are used and the method shall be such as to insure that each unit is uniformly wetted. All units shall be free from water adhering to their surfaces when they are placed in the wall. During freezing weather, units that require wetting shall be sprinkled with warm water just before laying.

(f) Protection of Work. During erection all walls shall be kept dry by covering at the end of each day or shutdown period with a canvas or waterproof paper. Partially completed walls not being worked on shall be similarly protected at all times. Covering shall overhang at least 2 ft. on each side of the wall.

(g) Freezing Weather.

1. No masonry shall be laid when the temperature of the outside air is below 40° F. unless suitable means are provided to heat the masonry materials and protect the completed work from freezing.

2. Protection shall consist of heating the masonry materials to at least 40° F. and maintaining an air temperature above 40° F. on both sides of the masonry for a period of at least 48 hr. if Type A-1 mortar is used and 72 hr. if Types A-2 or B mortars are used. These periods may be reduced to 24 and 48 hr., respectively, if high-early-strength cement is used.

(h) Joining of Work. Where fresh masonry joins masonry that is partially set or totally set, the exposed surface of the set masonry shall be cleaned, roughened, and lightly wetted so as to obtain the best possible bond with the new work. All loose brick and mortar shall be removed.

If it becomes necessary for construction purposes to "stop off" a horizontal run of masonry, this shall be done only by racking back ½ brick length in each course and, if grout is used, stopping the grout 4 in. back of the rack. Toothing will not be permitted except upon written approval of the architect.

In grouted construction when grouting is stopped for 1 hr. or longer, the grout pour shall be stopped 1½ in. below the top of the last course grouted except at the finishing course.

(i) Bonding.

1. Exposed brick and tile walls shall be laid in center bond or in the bond pattern indicated on the plans. In unexposed masonry, all vertical joints shall be bonded by at least 2 in.

2. All load-bearing masonry walls and partitions shall be bonded each course at corners and intersections and shall be bonded or anchored to connecting work. Non-load-bearing partitions shall be bonded or anchored as indicated on the plans.

(j) Jointing. When the mortar has become thumbprint hard, all exposed joints shall be tooled with a round or other approved jointer. The jointer shall be slightly larger than the width of the mortar joint so that a complete contact is made along the edges of the units, compressing and sealing the surface of the joint. Exterior joints below grade shall be trowel-pointed and all other joints shall be flush cut.

(k) Built-in Work. The masonry contractor shall build in all sash frames, louvres, cleanout doors, etc., provided and installed by other trades.

(l) Caulking. Outside joints at the perimeter of exterior door and window frames should be not less than ¼ in. nor more than ⅜ in. wide and shall be cleaned out to a uniform depth of at least ¾ in. and filled solidly with an approved elastic caulking compound forced into place with a pressure gun. Caulking compound shall be elastic and waterproof and shall gradually form a thin, tough skin on exposed surfaces while remaining plastic indefinitely beneath the surface. It shall not be affected by long exposure to extremes of outside temperature.

(m) Pointing and Cleaning.

1. At the completion of the work, all holes or defective mortar joints in exposed masonry shall be pointed and, where necessary, defective joints shall be cut out and repointed.

2. Exposed masonry shall be protected against staining from wall coverings or other sources and excess mortar shall be wiped off the surface as the work progresses.

3. At the completion of the work, all exposed masonry shall be thoroughly cleaned. If stiff brushes and water do not suffice, the surface shall be thoroughly wetted with clear water and then scrubbed with a solution of not more than 1 part hydrochloric (muriatic) acid to 9 parts water, followed immediately by a thorough rinsing with clear water. If masonry is cleaned with an acid solution, all sash, metal lintels and other corrodible parts shall be thoroughly protected.

Note 1. Green stain or efflorescence resulting from vanadium salts in the masonry must not be treated with an acid solution. To remove this stain, follow the recommendations of the manufacturer of the masonry units.

Note 2. Reinforced brick masonry floor slabs or other horizontal masonry surfaces which will be exposed to the weather should not be treated with acid solution, due to the absorption of the solution by the mortar joints which will

later result in efflorescence and possibly in disintegration of the mortar joints. It is recommended that the top surface of the brick to be used for grouted reinforced slabs be painted with a thin coating of paraffin oil or a thin sizing of clay. This will prevent adherence of the mortar or grout to the finished surface. After the mortar has hardened, the slab surface may be cleaned with water.

4. Acid solutions shall not be used for cleaning glazed units. Upon completion of the work, all surfaces of glazed wall units shall be washed with soap powder and warm water, applied with a scrubbing brush, and then rinsed thoroughly with clear water. Metal cleaning tools and brushes or abrasive powders shall not be used.

(n) Parging and Dampproofing.

1. The outside of basement walls in contact with the earth shall be parged with a ½-in. coat of Type A-1 mortar trowelled smooth, bevelled at top and coved out to the edge of the footing. This parging shall extend to 4 in. above finished grade, to the level of under side of area walls, and at least 12 in. on to area, porch and garage walls.

2. After parging has set for at least 7 days and when the surface is dry, it shall be covered with a dampproofing consisting of 1 coat of creosote oil and 2 coats of coal tar pitch. The pitch shall be heated to flow freely but not above 350°F. Dull or porous spots appearing after application shall be remopped with pitch.

703. REINFORCED BRICK MASONRY

(a) Types of Mortar and Grout. Mortar and grout for use in reinforced brick masonry shall conform to the requirements of Section 702(b) for Type —, (specify A-1 or A-2) except that only mortar and grout consisting of portland cement, hydrated lime or lime putty and aggregate shall be used.

(b) Reinforcement.

1. Reinforcement shall be accurately formed to the dimensions indicated on the drawings. All bars shall be bent cold.

2. Metal reinforcement shall not be straightened or repaired in a manner that will injure the material. Bars with kinks or bends not shown on the drawings shall not be used. Heating of the reinforcement will be permitted only when the entire operation is approved by the engineer.

3. Metal reinforcement shall be accurately positioned and rigidly secured against displacement from the location shown on the contract drawings. Where necessary, vertical reinforcement shall be held firmly in place by means of frames or other suitable devices. Horizontal reinforcement may be placed as the brickwork progresses.

4. The minimum clear distance between parallel bars, except in columns, shall be equal to the nominal diameter of the bar. In any space containing reinforcement, except small rods or mesh ¼ in. or less in diameter, the clear distance between any masonry and the reinforcement shall be not less than ¼ in. Vertical (collar) joints containing both horizontal and vertical reinforcement shall be not less than ½ in. wider than the sum of the diameters of the horizontal and vertical reinforcement contained therein.

5. In splicing vertical reinforcement or attaching to dowels, the bars shall be placed in contact and wired. Splices in adjacent horizontal bars shall be staggered. When it is necessary to splice reinforcement at points other than

shown on the drawings, the character of the splice shall be approved by the engineer.

(c) Workmanship—Reinforced Grouted Brick Masonry.

1. All reinforced grouted brick masonry shall be laid plumb and true to line and no bonding headers shall be used unless shown on the contract drawings.

2. All brick masonry shall be laid with full head and bed joints. The ends of brick shall be buttered with sufficient mortar to fill the head joint and furrowing of bed joints will not be permitted. The top of the bed joint mortar may be sloped toward the center of the wall to minimize the amount of mortar forced into the grout space when the brick is shoved to line. Mortar protruding from bed joints into grout space shall be removed before pouring the grout.

3. One of the outer tiers of masonry may be built up not more than 3 courses before grouting, but the other shall be built up not more than 1 course above the grout. Each pour of grout shall be stopped at least 1½ in. below the top and puddled with a wooden puddler. Slicing with a trowel is not acceptable.

4. In masonry more than 2 brick in thickness, the inner wythes shall be placed or floated in grout poured between the 2 outer wythes (tiers) with not less than ½ in. of grout between brick.

5. When pilasters or columns are built up with and are integral with the adjacent walls, grout shall be poured into the space surrounded by the boundary wythes of the brick in the pilaster and then the interior brick or bats placed in the grout in such a manner that the width of any vertical joint between brick does not exceed two times the width of the vertical collar joint of adjacent walls.

6. Wherever possible, grouting shall be done from the inside face of the masonry. Extreme care shall be used to prevent any grout or mortar from staining the face of masonry to be left exposed or painted. If any grout or mortar does contact the face of such masonry, it shall be immediately removed. Protect all sills, ledges, offsets, etc., from droppings of mortar and protect door jambs and corners from damage during construction.

(d) Workmanship—Bonded Reinforced Brick Masonry.

Bonded reinforced brick masonry shall be laid in accordance with Section 703(c), except that masonry bonding units or headers shall be included as provided in Section 704(c), or as indicated on the drawings.

(e) Forms and Shores.

1. Forms shall conform to the shape, lines and dimensions of the members as called for on the drawings, and shall be substantial and sufficiently tight to prevent leakage of mortar. They shall be properly braced or tied together so as to maintain position and shape.

2. In no case shall shores and forms be removed until it is certain that the masonry has hardened sufficiently to carry its own weight and all other reasonable temporary loads that may be placed on it during construction. The results of suitable control tests may be used as evidence that the masonry has attained such sufficient strength.

3. For girders and beams, the minimum time which shall elapse before removal of shores or forms shall be 10 days after the completion of the member, providing that suitable curing conditions have been obtained during that period. The forms and shores under slabs shall not be removed in less than 7 days after

completion of such slabs and then only when suitable curing conditions have obtained throughout the entire curing period. At least 16 hr. shall elapse after building masonry columns or walls before constructing floor or roof loading applied uniformly and an additional 48 hr. shall elapse before applying a concentrated load such as truss, girder or beam.

704. UNREINFORCED MASONRY

(a) Types of Mortar.

1. Type A-1 or A-2 mortar shall be used for all foundations and masonry in contact with earth.

2. Type A-1, A-2 or B mortar shall be used for all other exposed masonry and for load-bearing walls of hollow masonry units.

3. Type A-1, A-2, B or C mortar shall be used for all other masonry.

(b) Workmanship.

1. All masonry shall be laid plumb and true to lines.

2. Brick masonry shall be laid with full mortar joints. Mortar beds shall be spread smooth or only slightly furrowed. The ends of brick shall be buttered with sufficient mortar to fill the end joint. The vertical longitudinal joint in solid brick walls shall be completely filled by parging, by pouring the vertical joint full of grout, or by shoving. Closures shall be rocked into place with the head joints thrown against the two adjacent brick in place.

3. Horizontal cell structural clay tile units, exceeding 4 in. in thickness, shall be laid with divided bed joints and, where the units contain a drainage channel to conduct moisture to weep holes, this channel shall be kept free of mortar. Head joints of horizontal cell units shall be placed on both sides of the tile and sufficient mortar shall be used so that excess mortar will be squeezed out of the joints as the units are placed in position. Head joints may be buttered on both edges of the tile to be placed or one joint may be buttered on the tile in place and one on the opposite edge of the tile to be placed.

4. All vertical cell structural clay tile units shall be laid with divided head joints and, when such units are exposed in masonry walls, care shall be exercised to prevent continuous mortar joints through the wall.

5. All collar joints in exterior tile or brick and tile walls shall be completely filled with mortar. This shall be done by parging either the back of the facing or the face of the backing.

In laying brick and tile masonry, the mason shall avoid overplumbing and pounding of the corners and jambs to fit stretcher units after being set in position. Where an adjustment must be made after the mortar has started to harden, the mortar shall be removed and replaced with fresh mortar.

6. Non-bearing partitions, in locations where suspended ceilings do not occur, shall extend from the top of the structural floor to the bottom surface of the floor construction above and be wedged with small pieces of tile and the joint at the top shall be filled with mortar, unless otherwise shown on the drawings. In locations where suspended ceilings are used, non-bearing partitions shall extend to ———— (specify in accordance with building code requirements).

7. Where cutting of glazed units is necessary, the cuts shall be made with a motor driven masonry saw.

(c) Bonding.

1. Brick and composite brick and tile walls shall be bonded with masonry unit bonders (headers) or metal ties as specified or shown on the contact

drawings. When bonded with masonry units, not less than 4 per cent of the wall surface of each face shall be composed of bonders (headers) extending not less than 4 in. into the backing. The distance between adjacent bonders shall not exceed 24 in. either vertically or horizontally.

In solid masonry walls of more than 8 in. nominal thickness, the inner joints of bonding courses shall be covered with another bonding course which shall break joints with the course below.

When bonded with metal ties, the ties shall conform to the requirements of Section 704(c)5, and at least one such tie shall be provided for each 3 sq. ft. of wall surface. The distance between adjacent metal ties shall not exceed 24 in. either vertically or horizontally.

2. Multiple-unit tile walls shall be bonded through the wall at vertical intervals not exceeding 34 in. by lapping one unit at least 3½ in. over the unit below or by lapping with units at least 50 per cent greater in thickness than the units below at vertical intervals not exceeding 17 in.

3. Brick and tile veneer shall be attached to the backing either by masonry unit bonders as specified in Section 704(c)1 or by one corrosion-resistant metal tie for each 2 sq. ft. of wall area. The distance between adjacent metal ties shall not exceed 24 in. either vertically or horizontally. Metal wall ties shall be galvanized corrugated metal at least ⅞ in. wide and not lighter than U. S. Standard Gage No. 22, or other approved ties.

4. Multiple-unit non-bearing partitions may be bonded as specified for walls, paragraphs 1 and 2 of Section 704(c), or as specified for veneer, paragraph 3, Section 704(c).

5. Cavity walls shall be bonded with steel ties 3/16 in. in diameter, or metal ties of equivalent stiffness, coated with a non-corroding metal or other approved coating. At least one tie shall be used for each 3 sq. ft. of wall surface. The distance between adjacent ties shall not exceed 24 in. either vertically or horizontally.

Where hollow masonry units are laid with cells vertical, rectangular ties shall be used. In other walls Z-shaped ties shall be used. Rectangular ties shall be 6 in. long, approximately 2 in. wide, with ends lapped, and Z-shaped ties shall be 6 in. long, with 2-in. ends bent to 90° angles.

Ties shall be embedded in a horizontal joint of the facing and backing. Additional bonding ties shall be provided at all openings spaced not more than 3 ft. apart around the perimeter and within 12 in. of the opening.

(d) Anchorage.

1. When intersecting bearing walls are carried up separately, the perpendicular joint shall be regularly toothed or blocked with 8-in. maximum offsets and the joints provided with metal anchors having minimum cross-section of ¼ by 1½ in. with ends bent up at least 2 in., or with cross pins. Such anchors shall be at least 2 ft. long and the maximum spacing shall be 4 ft. Masonry walls abutting or adjoining the frame of a skeleton frame building shall be similarly anchored.

2. Non-bearing partitions, when anchored to abutting or intersecting walls or partitions, shall be anchored with metal ties or clips at least ⅞ in. wide and not less than No. 16 gage galvanized iron at vertical intervals not less than 4 ft.

3. Brick and tile facing against concrete shall be anchored to the concrete by the use of dovetailed anchors inserted in slots built into the concrete. Anchors shall be at least ⅞ in. wide and not less than No. 16 gage galvanized

iron. They shall be spaced not more than 18 in. vertically and 24 in. horizontally.

4. Two-inch split furring and 2-in. open back (split) glazed or unglazed facing tile shall be anchored to the backing with hardware cloth ties consisting of ½-in. mesh, No. 20 gage galvanized iron fabric, at least 4 in. wide and extending at least 3 in. into the masonry and to within ½ in. of the face of the furring, or by other approved ties. Ties shall be spaced not farther apart than 24 in. vertically and 36 in. horizontally.

5. Wood joists or wood beams bearing on masonry walls shall be securely anchored to the walls at intervals not exceeding 4 ft. by metal anchors having a minimum cross-section of 0.25 sq. in. and at least 16 in. long, securely fastened to the joists or beams at one end of the anchor by means of a single bolt or other approved method, and the other end of the anchor securely built not less than 3½ in. into the masonry.

Where joints run parallel to walls, such anchors shall be spaced at intervals not exceeding 6 ft. and shall engage not less than 3 joists, and the joists shall be solidly bridged at the anchors.

705. MATERIALS

(a) **Brick and Mortar.** ASTM Specifications C216- and C62- contain the requirement that grade MW brick shall have a minimum average compressive strength of 2500 psi. This grade (MW) is specified for exterior walls above grade.

As indicated in Chapter 3, allowable working stresses for reinforced masonry are taken as percentages of f'_m, the ultimate compressive strength of the masonry. This ultimate strength may be determined from tests of prisms as provided in Chapter 3, or may be assumed as a percentage of the compressive strength of the masonry unit, provided a specified mortar is used.

If f'_m is assumed to be 60 per cent of the brick strength with Type A-1 mortar and grout and 45 per cent of the brick strength with Type A-2 mortar and grout, working stresses for brick meeting the minimum requirements of the above ASTM specifications for grade MW would be based on masonry strengths (f'_m) of 1500 psi and 1124 psi for Types A-1 and A-2 mortars, respectively. If the design is based on higher working stresses requiring higher strength brick, the strength required should be included in the specifications. Also, if working stresses are based on ultimate strength of prisms, both the specifications for brick and for mortar should contain a strength requirement to insure that these strengths will be not less than the strengths of the brick and mortar used in constructing the prisms.

In grouted brick masonry construction, the absorption of the brick is relied upon to remove excess water from the grout prior to hardening, thus reducing the shrinkage and producing a grout mixture of low water-cement ratio. The removal of this excess water is essential to a strong bond between grout and brick and it also materially increases the strength of the masonry. For this reason, highly impervious units (5-hr. boil absorptions not over 2 per cent) are not recommended for both the facing and backing of grouted brick masonry walls, and for such construction it is recommended the specifications provide that masonry units in either the facing or backing, but not necessarily in both, at the time of laying shall absorb in 24-hr. cold immersion an amount of water weighing at least 5 per cent of the dry weight of the unit.

(b) **Metal Reinforcement.** The 1940 Report of the Joint Committee on Standard Specifications for Concrete and Reinforced Concrete makes the following statement regarding metal reinforcement:

"The principal requirements in the selection of metal reinforcement are the yield point, the bond value and the ductility.

"The advantage of high yield point steel has been recognized for many years, and is self-evident in view of the fact that the yield point of the reinforcement practically determines the ultimate strength of both flexural and compression members.

"In ordinary commercial grades of steel, a high yield point is generally accompanied by a decrease in ductility. The balance, therefore, between a high yield point and satisfactory ductility has developed the difference of opinion that exists as to the grade of steel to be used. The relative importance of these qualities depends on the character and service conditions of the structure. The choice of the grade of steel, therefore, becomes a matter of engineering judgment."

The great range between the yield point and the ultimate strength of a ductile material makes possible the absorption of a large amount of energy. This property is especially desirable where forces induced by earthquake or blast must be resisted without collapse.

Ductility in metal reinforcement is also required to permit fabrication by cold bending as required in the specifications. Metal that complies with the cold bend test of the applicable specification is sufficiently ductile for fabrication purposes.

The Joint Committee points out the need for "higher and dependable bond resistance than that obtained by the use of plain bars" and notes that there is no "standard specification for acceptable types of deformed bars."

Since publication of the Joint Committee Report, ASTM Standard Specifications for Minimum Requirements for the Deformation of Deformed Bars for Concrete Reinforcement, A305-, has been adopted and bars meeting the requirements of this specification are now generally available. The use of bars meeting this specification is recommended and only such bars are permitted the working stresses in bond recommended in Chapter 3 for deformed bars.

CHAPTER 8

CONSTRUCTION OF REINFORCED MASONRY

801. INTRODUCTION

The following sections supplement and amplify the specifications of Chapter 7 and contain recommended construction methods covering cold weather construction, cleaning, painting, the application of stucco and plaster, and the repair and maintenance of masonry walls. A section is also included on scaffolds. These recommendations are the result of numerous interviews with contractors and builders as well as a study of the data available from laboratory tests.

802. MATERIALS

(a) Acceptance. Evidence that all masonry units, mortar materials and reinforcement meet the requirements of the specifications should be obtained well in advance of starting the work. In many instances suppliers can furnish reports of tests indicating compliance of their products with specification requirements. If laboratory tests are required, samples for testing should be obtained well in advance of the shipping dates for the materials. A minimum of 10 days should be allowed the laboratory for completing and reporting tests on masonry units and mortar. If masonry prisms are to be built and tested, a period of approximately 40 days should be allowed, since the prisms must age for 28 days before testing.

(b) Receiving and Storing. In order to expedite the work, it is very important to check the shapes, sizes and quantities as soon as they are received on the job. An itemized list of materials should be prepared from the original estimate and deductions made from each item as the deliveries are received. To provide insurance against job delays, any shortages or omissions should be reported at once so that replacements can be made.

On large projects, the estimates should be made so that the requirements for a single building unit may be obtained directly from the summary list. In many cases it is very desirable to have the materials itemized by rooms so that delivery can be made to the most accessible locations.

When stored on the outside, all masonry units and reinforcement should be piled on plank platforms, preferably in a high, dry location. The various sizes and shapes should be placed in separate piles and in the proper order for use as needed on the job. During freezing weather, all material should be covered with tarpaulins or building paper. Glazed units should be covered at all times.

803. WORKMANSHIP

(a) Layout. It is good practice to lay the first course of units experimentally without mortar to determine the width of head joint required, so that the units may be laid without cutting. On short runs and narrow piers, however, some cutting and fitting may be required unless the job is laid out in multiples of unit lengths plus the mortar joint. With the development of modular building materials and general acceptance of modular coordination by

architects and engineers in the design of buildings, the use of modular masonry units will go far toward eliminating expensive adjustments of this nature, so often necessary in the past.

(b) Mortar. After the mortar materials and mix have been determined, provision must be made to maintain the correct batch proportions throughout the entire job. The use of corrected measuring boxes is recommended, although this method is not in general use. At the start of the job it is very important, however, to determine the amount of dry ingredients per cubic foot of material as received. Since bag batches are generally used, the amount of portland cement and dry hydrate are readily controlled. The quantity of dry sand and lime in putty, on the other hand, must be carefully determined.

Sand, as delivered on the job, contains a certain amount of moisture which causes it to swell or "bulk". If volume proportions are used without consideration of this characteristic, the mix will result in a shortage of sand. If the proportioning is done by counting the number of shovels, the pyramiding of moist sand may cause over-sanding. Eighty pounds of dry sand is considered equivalent to a cubic foot for volume proportions. After the weight of dry sand in a given volume of damp sand is determined, a fairly accurate control of the amount of sand may be had by filling a wheelbarrow to a predetermined level. Adjustment must be made, however, as a change of the moisture content of the sand is noted.

(c) Laying Brick. To secure the desired performance of brick masonry, good workmanship is essential and should not be sacrificed for speed of production. A skilled artisan will, however, produce good workmanship at relatively high speed, for his knowledge of the essentials automatically guides his technique.

In the National Bureau of Standards investigation of wall strength (RP108), two classes of workmanship were employed to determine their effect upon wall strength, and similar procedure was followed in other tests of moisture penetration. These two classes were called "Ordinary" (Type B) and "Inspected" (Type A) workmanship, and are described as follows:

"Ordinary" workmanship may be considered as that very closely approximating the quality of work generally obtained in commercial construction where close supervision is not obtained and the first consideration is speed. This class of workmanship is characterized by the furrowing or grooving of mortar beds very little mortar in the end joints, and almost no mortar in the collar (vertical longitudinal) joints. This type of workmanship resulted in lower strength and permeable (leaky) walls.

"Inspected" workmanship is that which may be obtained under a definite specification, followed by careful supervision. In this class of workmanship, the mortar beds are spread smooth or only slightly furrowed; the ends of the brick are buttered with sufficient mortar to completely fill the end joint when the brick is in place, and the vertical longitudinal joints are completely filled by pouring full of grout, by the "pick and dip" method or by "shoving"; the object to obtain a solid masonry mass entirely free from voids or open joints. This type of workmanship resulted in high strength (both compressive and transverse) and watertight walls. Walls built with this type workmanship were approximately 70 per cent stronger than walls built with Type B workmanship.

Workmanship, similar to the Bureau of Standards "inspected", Type A is specified in Chapter 7, and for solid brick walls or reinforced brick masonry, grouted construction is recommended, since grouting largely eliminates the

human factor in filling the interior joints. In grouted construction, all joints (bed and head) should be filled solidly with mortar.

The procedures for constructing reinforced brick masonry specified in Chapter 7 are illustrated in Fig. 8-1.

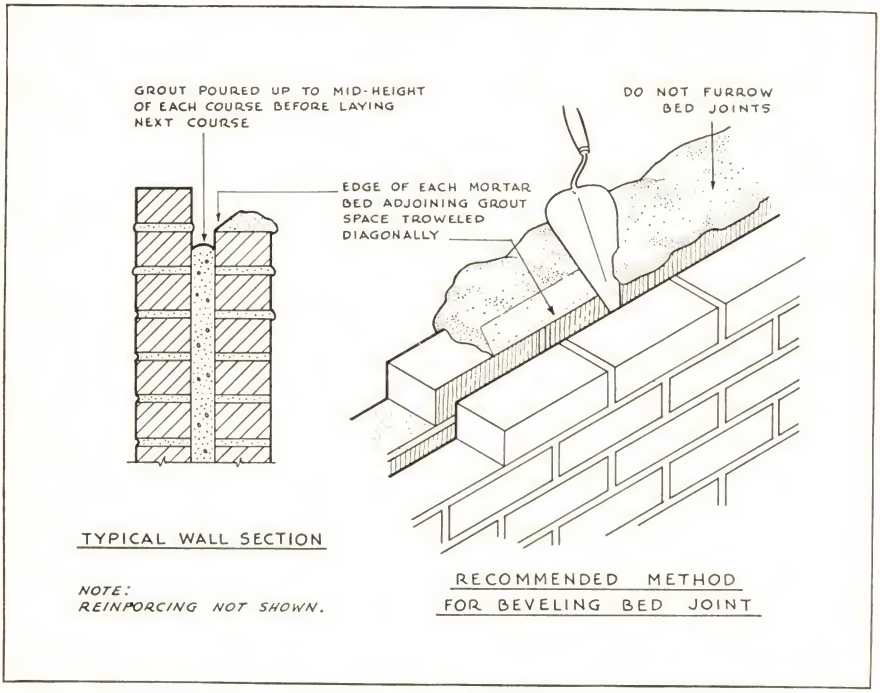


Fig. 8-1
Procedures for Constructing Reinforced Brick Masonry

Workmanship also includes other functions of great importance to the stability and appearance of brick structures. Brick should be laid on level beds and to plumb vertical lines. Interior spaces should be filled with grout and, where specified, wythes should be well bonded by headers, or as otherwise required. Special attention should be given to insure full head joints between header units. Architectural patterns and surface bonds should be carefully laid out in advance and carefully executed in the work, and horizontal and vertical joint thicknesses should be determined to work to openings and corners with the minimum of variation.

Bricklayers should, at all times, keep in mind the requirements of the other trades in order to build in or make proper provision for the installation of the other branches of the work. All such built-in work should be solidly bedded in mortar or grout.

(d) Covering. Proper covering of the wall in time of inclement weather is essential for good construction. Many fine buildings have been damaged by lack of precautions at such times. Covering of unfinished walls by tarpaulins or heavy waterproofed paper, securely tied or weighted in position, is specified in Chapter 7 and should be rigorously enforced. Mortar boards or scaffold boards should not be accepted as suitable covering.

(e) Reinforcement. The importance of protecting metal reinforcement in reinforced brick masonry from dampness will be obvious to all designers, Spalling of concrete and cracking of masonry due to corrosion of the reinforcement or metal ties has frequently caused serious damage to structures. In its 1940 report, the Joint Committee on Standard Specifications for Concrete and Reinforced Concrete states: "In structures exposed alternately to wetting and drying, this (corrosion) is probably the most common cause of deterioration."

The masonry protection recommended in Chapter 3 for reinforced masonry exposed to dampness is adequate to protect the reinforcement, provided there are no defects in the masonry.

Numerous tests at the National Bureau of Standards and other laboratories indicate that watertight brick masonry can be built if all mortar joints are well filled, the brick have a suction rate not exceeding ¾ oz. per min. when laid (reduced by wetting if necessary), and the mortar contains the maximum amount of water consistent with workability.

While it is important to observe these rules in constructing unreinforced masonry structures, they become doubly important in reinforced brick masonry construction, due to the requirements for high strength and protection of the reinforcement.

In reinforced brick masonry, the location of the reinforcement is fixed within narrow limits by the width of the joint between masonry units. However, in order to assure adequate bond between grout and reinforcement, it is important that the ¼-in. clearance specified between masonry unit and reinforcement be maintained. To insure this the vertical steel should be rigidly supported during construction.

(f) Reinforced Brick Lintels, Beams and Walls. Special soffit brick to provide space for reinforcing bars in lintels and beams are available in many localities or the space may be provided by the use of half brick (soaps).

In beams containing "Z" stirrups, the bars are spaced according to plan with the lower horizontal leg immediately under the longitudinal horizontal reinforcing rods. If the stirrups are long, they may be wired or tied to a board or rod at the top which serves as a template holding the stirrups firmly in place. This template is removed when the brickwork is carried up to within a course or two of the top horizontal leg of the stirrups. If the stirrups are short, the template or support at the top may be omitted because the first few courses of brick above the base leg of the stirrups will provide sufficient support to maintain the stirrups in the vertical position.

In reinforced brick walls, reinforcement is usually placed both horizontally and vertically. Special units may be used to form channels for this reinforcement or the bars may be placed in the longitudinal vertical (collar) joints. The vertical rods are placed and properly spaced by templates which are firmly supported. The templates may be boards with holes or slots at proper intervals or steel rods to which the vertical rods are wired.

It is not necessary to wire the vertical and horizontal reinforcing rods as in reinforced concrete. The horizontal rods need not be placed until the designated number of brick courses are built.

It is not always possible nor convenient to use rods long enough to extend the full length or height. Therefore, convenient lengths are used and the stresses transmitted from one rod to another by sufficient overlapping. The amount of overlap is determined by the calculated stress in the reinforcement and the allowable unit bond stress. This overlap will be shown on the drawings and, at laps of vertical bars, the rods should be securely wired together.

The same procedure should be followed in attaching vertical reinforcement to dowels in the foundation. It is most important that all bars and laps be placed as shown on the drawings.

(g) **Reinforced Brick Slabs.** Reinforced brick slabs may be precast or built in place. As a rule, their cost will exceed that of a reinforced concrete slab; however, where a brick surface is desired, the reinforced brick slab is frequently more economical. Procedures for building reinforced brick slabs, either precast or in place, are similar.

The brick for a single thickness slab are started along one edge of a slab and laid directly on the forms without bedding of any kind. For obtaining the desired width of mortar joints between brick courses, successful use has been made of a gas pipe, an electrical conduit or a smooth reinforcing rod of the proper size. After a row of brick is placed, the spacer (rod, pipe or conduit) is removed to the outside, ready for the next row. Some builders prefer to slush a small quantity of mortar down between the brick immediately after removing the spacer, or to dispense with spacers and spread some mortar on the side of each brick at the bottom edge before setting in place. The mortar is then tamped well in order to fill the bottom of the joint to a uniform height, depending upon the required location of the rods. This mortar tends to keep the brick in proper position, seals the bottom of the joint and serves as a bed for the reinforcing rods which are then dropped in place. If mortar is not placed in the bottom between the brick courses, wire hangers of the "U" type may be placed at proper intervals and the reinforcing rods in turn dropped into the open space between the brick, supported in the correct position by these hangers. After all reinforcement is in position, it is important to seal all exterior vertical joints by pointing with mortar before pouring the grout.

If the brick to be used for slab have high suctions (exceeding ¾ oz. per min.), they should be wet before using, as recommended for all masonry. It is also advantageous to paint the top surface of the brick with a thin coating of paraffin oil or with a thin sizing of clay. This will prevent the adherence of the grout to the finished surface and will result in a finished floor surface which will show the natural color and texture of the individual brick. After the grout has set, the finished slab surface is washed off with clear water. If the brick surface is to be ground and polished, it is not necessary to take the precaution of painting with oil or clay.

The grout used for the mortar joints is poured from a bucket or other suitable vessel directly into the open joints and should be just fluid enough to run freely around the steel and fill the joint. Grout is subject to considerable shrinkage and may tend to congeal and not flow completely into all the voids. Therefore, after a joint has been filled once, it will be necessary to run a trowel or pencil rod along the joint to aid the flow of grout and probably go back a second time to fill in the space caused by such shrinkage.

If a slab of two courses of brick is to be built, proceed as outlined above; then spread a bed of mortar for the middle or horizontal joint, using wood or pipe screeds of specified thickness for leveling off. Into this horizontal mortar joint place all transverse rods. The top course of brick is then laid in the same manner as described above and reinforced as required by the plans.

A very beautiful floor finish may be obtained by filling the uppermost ½ in. or ¾ in. of the mortar joints with a terrazzo floor mixture and then grinding, filling and waxing. This terrazzo mixture may be had in a variety of colors. A basketweave bond pattern for the brick slab lends itself to easy

placement of reinforcement, particularly where two-way reinforcement is required. The usual running bond pattern in a one-course slab can be used for one-way reinforcement only.

(h) Reinforced Brick Columns. In building reinforced brick masonry columns, it is important that dowels of proper length be placed accurately in the footings. After the footings are completed, the vertical bars in the column are securely wired to the dowels, and frequently a wood template is used to hold the rods at the top. Before the uppermost ends of the rods are tied or braced in place, it is preferable that the required number of hoops be slipped over the top of the vertical bars. This is essential if the hoops are welded. These hoops are held temporarily near the top until removed as needed for placement in the respective mortar beds. It is not necessary to wire the hoops to the longitudinal reinforcement, but care should be observed in placing each hoop so that it is centered on the column section and the core of the column is straight and plumb.

The brickwork is similar in character to other forms of reinforced brick masonry. A full bed of mortar is spread evenly, the lateral reinforcement embedded therein and the mortar again trowelled smooth before the next course of brick is laid. The brick are placed firmly in the mortar, usually by shoving, with all vertical joints filled solidly with mortar. Brick bats or large pieces may be used inside or around the vertical reinforcement wherever it is impossible or inconvenient to lay full size brick. Tests prove that these bats, surrounded by grout, develop full strength of the masonry.

After all brick are laid in a course, with all exterior vertical joints sealed with mortar, grout is poured so as to fill completely all remaining space. The large pieces or bats of brick are then "floated" or placed in the grout and tapped or pressed firmly down to insure their proper embedment. Some precaution is necessary to insure complete filling around reinforcing bars. The next bed of mortar is then spread and the process repeated.

Sometimes, the inner core is filled with a rich concrete. In such cases, the brickwork is completed several hours, and preferably a whole day, before the concrete is poured into the core. Needless to say, the concrete should be well spaded to insure complete filling of voids and complete encasement of the reinforcement.

804. FORMS

Form are not required for vertical reinforced brick members, such as walls and columns. This is one of the principal factors which contributes to the economy of this type of construction. Forms to support the soffit of beams and decking for reinforced brick slabs are required and can be made of any material possessing sufficient strength to carry the superimposed load. The thickness will, of course, be governed by the spacing of the supporting members. With a suitable spacing of joists, ⅝-in. plywood oiled to prevent the absorption of moisture will be found to give excellent results.

Forms should be clean when they are used, free from ice or snow, and must be properly braced and tied together so as to maintain position and shape.

805. WETTING BRICK

Laboratory tests as well as the performance of brick and tile walls in service indicate that a strong and complete bond cannot be obtained between mortar and high absorption brick and tile units laid dry. Good mortar bond

is essential to both strength of masonry and resistance to rain penetration, and such a bond can be obtained with units of high absorption if they are properly wetted before laying.

A rough but effective test for determining what units give improved bond by wetting consists in drawing a circle 1 in. in diameter on the surface of the unit which will be in contact with the mortar with a wax pencil, using a 25-cent piece as a guide. With a medicine dropper, place 20 drops of water inside this circle and note the time required for the water to be absorbed. If the time exceeds 1½ min., the unit need not be wetted; if less than 1½ min., wetting is recommended.

A satisfactory procedure for wetting brick and tile consists of playing a stream of water on the pile until water runs from each individual unit. This should be done several hours before the units will be used so that the surfaces will have an opportunity to dry before they are laid. Wetting of low absorption units or the excessive wetting of other units which results in water on the faces when they are laid is undesirable and will cause "floating" and bleeding of the mortar.

806. BONDING

Both the strength and appearance of masonry walls are affected by the bonding of units. Longitudinal bonds and patterns should be laid out in advance of the work and checked carefully as the work progresses. Relatively slight variations in patterns may destroy the desired effect.

Through-the-wall or structural bonding units (headers or bonding stretchers) should be bedded in full mortar joints and particular care should be exercised to obtain full head joints since wall leakage frequently occurs at header courses.

Corners of brick and tile walls should be bonded every course and no unit less than 4 in., horizontal face dimension, should be used at a corner or jamb.

807. MISCELLANEOUS CONSTRUCTION METHODS

(a) Nailing Plugs, Blocks and Anchors. Wood nailing blocks used for fastening furring, grounds, picture molds or other surface fixtures should be of seasoned soft wood creosoted to prevent shrinkage and rot. They should be placed in the vertical joints only and never in the horizontal bed.

Metal nailing plugs are recommended as providing better construction. They may be built into the joint when the wall is erected, but it requires additional care to locate them exactly where required. This is not a serious problem when used for baseboards, chair rails, or picture molds; however, it may be difficult to determine the location for plumbing fixtures, cabinets and shelving. With layouts incorporating modular coordination on the 4-in. increment, the exact location for inserts of this type can be readily obtained—in addition, fixture sizes will be standardized on the 4-in. increment so that spacing will be uniform for the various makes.

(b) Furring Applications. Although there are many examples of masonry walls with plaster finish applied directly to the interior surface, furring on 8-in. walls is recommended in northern areas, particularly for residential construction. Furring may be of wood, hollow tile or metal, depending upon the type of construction and the local building requirements.

Wood furring generally consists of 1 by 2-in. or 2 by 2-in. strips, applied vertically to the face of the wall by nailing into blocks, wood or metal plug inserts, or directly into the mortar joint by the use of case-hardened "cut" nails or special spiral-threaded masonry nails.

Furring strips may be fastened to tile walls with expanding or "self-clinching" nails, driven into the end joints, or collapsible steel screw sockets may be used. Special anchor nails fastened to the masonry wall with adhesive cement is a recent development for installing furring and plaster grounds. They are easily and quickly installed without drilling, plugging or nailing. Spacing of furring strips is determined by the width and thickness of the lath, plaster or insulation board, but is usually 16 in. on center. Several methods of attaching wood furring strips are illustrated in Fig. 8-2.

Metal furring strips consist of standard light steel channels fastened by either tie wires built into the mortar joints or by special clips designed for this purpose.

Tile furring may be free-standing or anchored to the walls with metal ties or 10d nails driven into the joints and bent over the top edge. Split furring tile, 2 in. in thickness and 12 in. square, are generally used. For this size, the ties are spaced every two courses in height and not farther apart than 3 ft. horizontally. The use of furring tile is described and illustrated in Chapter 10 of Brick and Tile Engineering, published by the Structural Clay Products Institute, 1950.

(c) Framing at Openings. Door and window openings which are to receive trim must be provided with means of fastening to the masonry. This usually consists of a framing around the opening, commonly called a "rough-buck," and may be of wood or metal construction. Door bucks are set in the exact position as shown on the plans and anchored to the floor and temporarily braced in a true position. As the masonry is built up solidly against the buck, metal anchors are attached to the buck and bedded into the mortar bed.

Combination door bucks and jambs of pressed metal are often used with masonry partitions. Corrugated adjustable anchors are furnished for approximately each 2 ft. of opening height. For heavy-duty openings, steel channel frames are generally used.

(d) Cutting. During the past several years rapid advancements have been made in tools with which to cut structural clay products. Specially designed masonry saws are now used to provide precision angles, outlets and shorter lengths on the job or in the plant.

Regular wood-cutting saws should not be used for masonry work for several reasons. The motors must be totally enclosed to withstand abrasions; moreover, a different cutting principle is necessary for masonry materials. Masonry saws utilize a ⅛-in. thick silicon-carbide blade, bonded with resonoid. For full efficiency the blade must not be jammed into the unit, but instead the material being cut should be moved back and forth beneath the blade, completing a rapid series of cuts. This principle of operation lessens the arc of contact on the blade which is absolutely necessary to assure the complete elimination of cutaway particles, thereby greatly increasing blade life.

Actually, by test, it is found that a masonry saw equipped with a 1½ hp motor will complete a cut through a 4-in. glazed brick in 22 to 32 sec., while the jam or wood-cutting principle will greatly reduce the cutting speed as well as decrease blade life.

It is generally best to cut completely through a unit, rather than to score and break, taking into consideration the time required to pick up a hammer and chisel and final trimming necessary. Then, too, considerable material can be salvaged when a portion of the unit is sliced away neatly and accurately.

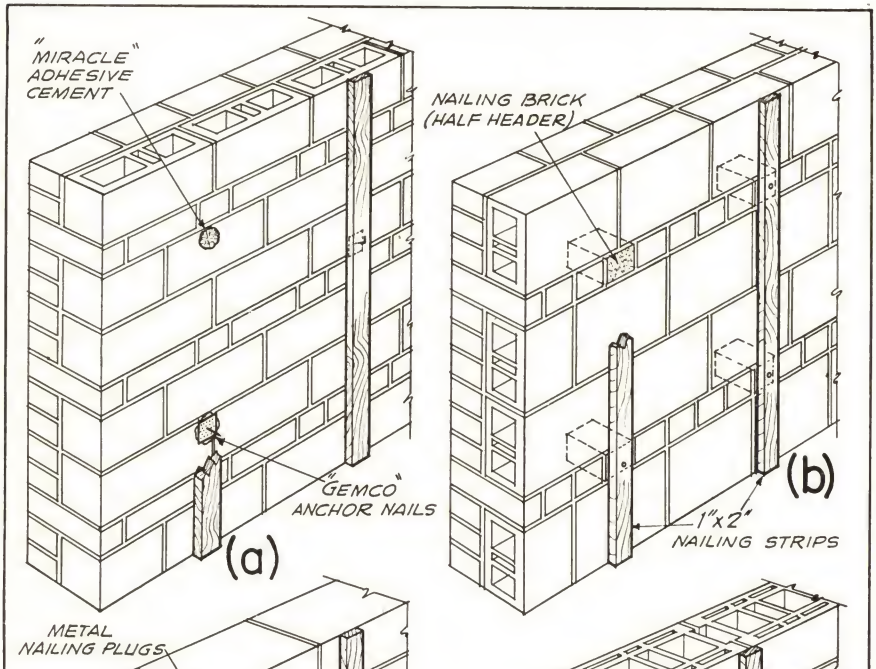


Fig. 8-2

Typical Methods of Attaching Wood Furring Strips to Masonry Walls

Masonry saws are designed to afford maximum flexibility in handling furring tile as well as the largest clay unit. Foot pedal operation is recommended to give the operator full use of both hands in order to mark the unit and perform the cutting.

808. COLD WEATHER MASONRY CONSTRUCTION

(a) Discussion. In many localities within comparatively recent years, it was considered good practice to discontinue all masonry construction during the severe winter months. Most projects were planned so that this portion of the work could be done in the summer and early fall, thus creating an unusual demand for workers during these months which was followed by a long period of unemployment. To a certain extent, this was justified when slow setting lime mortars were used. The use of these mortars necessitatel long protective periods for the masonry walls and since the protective methods and equipment in use at that time were costly, only a few of the larger contractors were properly equipped to carry on winter masonry construction.

However, mortars containing portland cement in varying amounts have come into almost universal use and the shorter setting time of these mortars has substantially reduced the protective periods required for masonry walls built in sub-freezing temperatures. Protective methods and equipment have been improved until brick and tile masonry may now be constructed economically throughout the year in nearly every section of the United States.

While the cost of protective measures required to prevent masonry from freezing before it has acquired sufficient strength to resist the loads to which it may be subjected, and to insure its durability, is an additional item of expense over the cost of summer construction, there are other compensating items which will frequently make the cost of winter construction no greater, and sometimes less than the cost of similar work constructed during the summer months. The most important of these compensating items is the greater availability of labor and materials during the "off-season." Several contractors report that the production of their masons during the winter months exceeds the average production during the summer and the yearly labor costs of the contractors who operate on a yearly basis is substantially less than if work is carried on only during the warmer months. Building materials can usually be obtained at somewhat reduced prices during the winter because of the smaller demand and the desire on the part of supply dealers and manufacturers to keep their men employed. Shipping facilities are less crowded during the winter months and deliveries can, as a rule, be made more promptly.

(b) General Requirements. Since jobs vary as to size, layout, height, general design, location in respect to adjoining structures and in many other respects, the most economical methods of protecting and heating the particular project should be studied before detailed procedures are established. In general, the items to be considered in the protection of masonry construction in sub-freezing weather are the following:

1. Proper storing of materials. All masonry units and mortar materials should be placed on plank platforms and thoroughly covered with tarpaulins or building paper. Planks should be raised to prevent absorption of moisture from the ground. Masonry units and mortar materials should never be permitted to become coated with ice or snow. Carelessness in the storing of materials promotes poor workmanship and increases the cost of laying, as the removal of ice and snow, and the thawing of masonry units are absolutely

necessary before construction is started. Since the cost of acceptable covers is no real burden to the job, specifications for cold weather masonry should include the use of such platforms and covers, and forbid the use of unprotected materials exposed to the weather.

2. Preparation of Mortar. Batch concrete mixers for mixing all types of mortar are recommended on all large jobs and they are sometimes economical on smaller jobs. Modern mixers are equipped with a skip hoist, water tank and water measuring device which control the mix and produce well-mixed mortar of the right workability. Steel mortar boxes may be used on smaller jobs. The boxes should be raised about 1 ft. above the ground on piers built of brick laid dry. Waste building wood or steam may be used to keep the mortar warm after mixing.

The mixing water and sand should be heated not to exceed 160° F. The mortar sand should be heated uniformly. Scorched sand (with a reddish cast) should never be used in the mortar. Scorching can be prevented by heating slowly and evenly. This can be done by piling the sand around an old metal smoke stack section laid horizontally, in which a slow fire is built, or through which steam pipes may be run. In freezing weather, both the water and sand should be heated.

Temperature of the mortar when used should not exceed 120° F. nor be lower than 70° F. The use of salt to lower the freezing point of the mortar should not be permitted.

3. Heating the Materials. Heating of masonry units is also recommended in severe weather. To prevent sudden cooling of the warm mortar in contact with cold units, it is recommended that all masonry units be heated when the outside temperature is below 40° F. Masonry unit heating methods should be given careful study and, where required, storage should be provided so that heating may be facilitated at minimum expense.

Brick having rates of absorption above ¾ oz. per min. should be wet with warm water just before laying; those with lower rates of absorption can be laid dry.

4. Laying Precaution. Masonry should never be placed on a snow or ice-covered base or bed because of the danger of movement when the base thaws and the total lack of bond between the mortar and bed. If the walls are properly covered at the end of the work period or in preparation for a "shut-down" period, there should be no necessity for ice or snow removal from walls. In the event, however, that the covering is displaced, the bed may be cleaned with live steam. The steam should be applied long enough to thoroughly dry out these parts and, if frozen or damaged, defective parts should be replaced before starting new brickwork.

5. Protection of Walls. Protection of the masonry is required and will vary with the weather conditions. With warm masonry materials, tarpaulins covering the masonry may be sufficient when temperatures are above 25° F. on a rising thermometer. In severe weather, however, enclosures with artificial heat by steam or coke-burning salamanders are recommended. Walls should not be baked on one side with no protection on the other, but the enclosures should be so arranged as to allow a circulation of warm air on both sides of the wall. Each job is an individual problem. Job layout, desired rate of construction and the prevailing weather conditions all influence the amount of protection and the amount of heat necessary to prevent the masonry from freezing. Fig. 8-3 shows an effective method of protecting masonry walls.

Fig. 8-3

(Top) Typical Coke-burning Salamander Used to Heat Enclosures During Cold Weather Construction. (Bottom) Partial Canvas Enclosure Protecting Masonry and Workmen During Cold Weather Construction

When a building is enclosed and only partitions and inside walls are being constructed, the matter of protection reduces itself to the closing of all door and window openings (before glazing) with heavy muslin.

A further precaution would be the closing of all stair wells and elevator shafts. After taking these precautions, a few well placed "salamanders" should produce the required temporary heat.

809. CLEANING CLAY PRODUCTS MASONRY

(a) New Construction. In the construction of masonry walls the skilled mason will generally keep the surface remarkably free from mortar particles and stains, being trained to take pride in the appearance of his work. There are few other occupations where the individual's character is so readily reflected. At the end of each day, the mason's performance is exposed not only to the critical eyes of the fellow workmen and foremen, but also to the builder and owner. In addition, the completed portion of the wall becomes an important part of the finished structure always exposed to the public view.

One of the important techniques acquired by the mason is the "feel" of or adeptness with the trowel. The proper amount of mortar is taken up each time, until this becomes an automatic operation. It is then carefully placed to prevent the excess mortar from spreading or dropping on the face of the wall as pressure is exerted on the unit. Greater care must be taken when laying rough textured units; also when using the rich cement mortars since they are characteristically less plastic and may have lower water retentivity. With hard unglazed units, bleeding may cause some staining, and particles splashed on smooth unglazed surfaces will leave temporary marks. For this reason most specifications require a final washing down of all masonry work. Typical methods and requirements are as follows:

1. Unglazed Masonry Surface. On completion of the work all masonry must be carefully cleaned down, removing all large particles of mortar with a putty knife or chisel. If acid is required for the removal of mortar stains (see note below), it should be muriatic (hydrochloric) and not stronger than 1 volume of the commercial acid to 9 volumes of water. Before the acid solution is applied, the surface should be thoruoghly soaked with clear water, otherwise the mortar stain may be drawn into the pores causing a permanent dulling of the natural masonry colors. The acid solution should be applied with a long-handled, stiff fiber brush, with proper precautions as to protection of clothing, hands, and eyes to prevent burns. It should not be placed over an area greater than 15 to 20 sq. ft. before the wall is again thoroughly washed down, or preferably hosed, with clear water immediately after cleaning. It is important to remove all trace of the acid before it attacks the mortar joint. All frames, trim, sills, or other installations adjacent to the masonry must be carefully protected against contact with the acid solution.

Note: Whenever possible, smooth, light-colored units should be scrubbed with warm water and soap powder in lieu of acid cleaning.

2. Glazed Surface. Acid cleaning is not required for glazed masonry units. As the work progresses, any excess mortar should be removed with a cloth. Upon completion of the work, all surfaces of glazed wall units may be cleaned down using soap powder and warm water, applied with a fiber scrubbing brush, and then rinsed thoroughly with clean water. Hard lumps of mortar may be removed by using sharpened wood paddles. Metal cleaning tools and brushes or abrasive powders should not be used.

(b) **Old Construction.** Processes used in cleaning depend primarily upon the materials to be cleaned and the nature and chemical composition of the spots or stains to be removed. Cleaning clay products is no exception to this general rule. A method which can be used successfully with glazed ware, for instance, may prove entirely unsatisfactory if applied to a rough textured masonry wall. The problem is further complicated by the fact that masonry structures consist of two materials, the clay units and the mortar, each of which has different characteristics—many cleaning compounds to which clay products are entirely impervious have a damaging effect upon mortar.

Texture and absorption* of the clay unit and the nature of the material or stain to be removed also determine to a very great extent the results which may be obtained with various cleaning methods. Proper cleaning methods will restore glazed ware to its original appearance. The same is true of smooth textured brick or tile of relative low absorption; the lower the absorption the better the results which may be expected.

High absorption or rough textured units are more difficult to clean. If the staining material has penetrated into the pores of the clay, it frequently cannot be removed without removing a part of the unit itself, thus destroying the original texture and resulting in an appearance different from the original.

Principal methods of cleaning masonry structures are: sand blasting, steam or steam and water jet and the application of various cleaning compounds. The first two of these methods are used principally on large buildings as considerable equipment is necessary for either method. Cleaning compounds may be applied to either large or small structures.

1. Sand Blasting. Sand blasting consists in blowing hard sand through a nozzle by compressed air against the surface to be cleaned. The sand removes a thin layer from the wall surface, the thickness of the layer depending upon the depth to which dirt or stain may have penetrated the wall. This method is an effective cleaning process; however, it destroys the original texture of the unit and leaves the wall with a coarse texture which is particularly susceptible to the accumulation of soot and dirt. Due to the difference in hardness between clay units and the mortar joints, sand blasting may do serious damage to the joints.

If sand blasting is used, it will frequently be necessary to repoint the mortar joints after the surface has been cleaned; and the application of a colorless waterproofing compound to the roughened surface will tend to make the wall self-cleansing, and will prevent the rapid soiling of the surface from smoke and dust particles in the air. Sand blasting should never be used on glazed ware or other units having special surfaces or textures.

2. Steam or Steam and Water Jets. This method of cleaning consists of washing the wall with a steam or steam and water jet under pressure. It is effective in removing soot and dirt which accumulates on a wall over a period of time. Best results are obtained when it is used on glazed ware or low absorption units; however, it has also been used with fair success on high absorption or rough textured units. It is not effective in removing stains which have penetrated into the pores nor in removing such substances as mortar or paint.

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* Porosity, absorption, capillarity and rate of absorption are all factors which affect the cleaning of clay products. In this discussion the term absorption should be understood to include all the above.

Frequently an alkaline such as sodium carbonate, sodium bicarbonate or trisodium phosphate is added to the cleaning water when the cleaning is done by the jet method. Chemically these compounds are known as salts and, while they aid materially in the cleaning action, some of the salt will be retained in the clay unit (the amount depending largely upon the absorption of the unit) and will appear later on the face of the wall in the form of efflorescence. The amount of the salt retained by the clay units can be materially reduced by thoroughly wetting the surface with clear water before the cleaning solution is applied. The wall should also be washed with an abundance of clear water after cleaning to remove the salt from the surface.

3. Application of Cleaning Compound. This method is applicable to structures of all sizes and is probably used to a greater extent than any of the other methods. On large projects the cleaning contractor develops a cleaning compound best adapted to the particular job. This is done through an examination and analysis of the material and stains to be removed and the strength and chemical composition of the solution is usually adjusted by trial.

4. Treatment of Surface Prior to Cleaning. Most cleaning solutions contain compounds which if absorbed by the clay units will subsequently appear on the face of the wall in the form of efflorescence. For this reason the surface to be cleaned should be thoroughly wetted with clear water before the cleaning solution is applied. A booklet,—"Maintenance of Interior Marble" by D. W. Kessler—which is based upon the results of over 10,000 laboratory tests and experiments, contains the following statement: "The importance of thorough rinsing is so evident and has been so frequently stressed that it hardly seems worth while to emphasize the point here. However, the research upon which many of the recommendations herein are based has indicated that the final rinsing is no more important than the preliminary dampening of the surface with clear water. Where this practice is followed the final rinsing can be more easily and satisfactorily done."

5. Removing Efflorescence. Water applied with stiff scrubbing brushes will frequently remove efflorescence. If this does not do a complete job, apply water first, then scrub with water containing not more than 10 per cent of muriatic (commercial hydrochloric) acid and, immediately thereafter, rinse thoroughly with clean water. It is sometimes desirable to give the surface a final washing with water containing approximately 5 percent of household ammonia.

Note: Green stain or efflorescence resulting from vanadium salts in the masonry must not be treated with an acid solution. To remove this stain, follow the recommendations of the manufacturer of the masonry units.

6. Treatment for Removal of Stains.

Mortar Stains: The method described for washing down unglazed masonry wall surfaces after construction is also used for the removal of mortar particles and stains on existing work.

Paint Stains: For fresh paint apply a commercial paint remover, or a solution of trisodium phosphate in water—2 lb. of trisodium phosphate to 1 gal. of water. Allow to stand and remove paint with scraper and wire brush. Wash with clear water.

For very old dried paint, organic solvents similar to the above may not be effective, in which case the paint must be removed by sand blasting or scrubbing with steel wool.

The following recommendations are taken from "Maintenance of Interior Marble" by D. W. Kessler, or "Removing Stains from Cast Stone and Concrete" —Concrete Building and Concrete Products Vol. 12, No. 3, March 1937 by R. E. Baumgarten. It is believed that they may also prove effective with clay masonry.

Iron Stains: Mix 7 parts lime-free glycerine with a solution of 1 part sodium citrate in 6 parts lukewarm water, and mix with whiting or kieselguhr to make a thick paste. Apply paste to stain with trowel, and scrape off when dried out. Repeat until stain has disappeared and wash thoroughly with clean water.—R. E. Baumgarten.

Tobacco Stains: Dissolve 2 lb. trisodium phosphate in 5 qt. of water. In separate vessel make a smooth stiff paste of 12 oz. of chloride of lime in water. Pour the former onto the paste and stir thoroughly. When the lime has settled, draw off the clear liquid and dilute with equal parts of water. Make a stiff paste of this with powdered talc and apply in the same way as described for iron stains.—R. E. Baumgarten.

Smoke Stains: Make smooth stiff paste of trichlorethylene and powdered talc and apply as described above. Cover with glass or pan to prevent rapid evaporation. If slight stain is left after several applications, wash thoroughly and then use the method described under tobacco stains. Precaution should be taken to ventilate a closed space in which trichlorethylene is used, as the fumes are harmful.—R. E. Baumgarten.

Copper or Bronze Stains: Mix together in the dry form, 1 part of ammonium chloride (sal ammoniac) and 4 parts of powdered talc. Add ammonia water and stir until a thick paste is obtained. Place this over the stain and leave until dry. When working on polished marble use a wooden paddle to scrape off the paste. An old stain of this kind may require several applications. Sometimes aluminum chloride is used in the above procedure instead of the sal ammoniac.—D. W. Kessler.

Oil Stains: Make a paste of a solution of 1 lb. of trisodium phosphate to 1 gal. of water and whiting which may be obtained at any paint store. Spread this paste in a layer about ½ in. thick over the surface to be cleaned and leave until it dries (approximately 24 hr.). Remove the paste and wash surface with clear water.—D. W. Kessler.

Coal Smoke Stains: Scrub with powdered pumice and wire brush. This is an alternate method to the one recommended by Mr. Baumgarten.

7. Glazed Ware. In cleaning glazed ware, care should be exercised to select a cleaning compound which will not erode or etch the glaze. Most cleaning preparations contain scouring grit, powdered soap and carbonate of soda. The per cent of scouring grit varies in different cleaners from 30 to 90 per cent and, as it is frequently harder than the glazed unit, it has a tendency to scratch the glaze. Before using a cleaning preparation it should be tried out on a small area to make sure that the scouring grit will not mar the finish. In using cleaning compounds which contain soap powder, soft water should be used or, if it is not obtainable, hard water should be softened. Trisodium phosphate which may be purchased from laundry supply houses in the form of a white powder may be used as a water softener.

Sodium hydrosulphite, acidified sodium fluosilicate, and ammonium bifluoride have been used effectively with glazed ware and if sufficiently diluted are not apt to etch the glaze. The cleaning solution should be prepared by adding ½ to ¾ lb. of any one of the above to 1 gal. of water. When using

these compounds all metal and glass should be protected from the cleaning solution. The cleaner should be mixed in wooden containers and in applying it rubber gloves should be used to protect the hands.

810. SCAFFOLDING AND TOWERS

(a) General. Scaffolding may be separated into two structural elements; (1) supporting members and (2) platforms. The former may consist either of typical wood construction or of built-up sectional steel tubing, while the platform or "staging" is generally nominal 2 by 10-in. planking for all types.

During the past decade, tubular steel sections have been extensively used for exterior and interior scaffolding on all classes of buildings. They are especially practicable for construction where the fire hazard from flammable scaffolding may be very serious. In some localities, all lumber used for scaffolding must be processed with chemicals to render it fire resistive.

For high skeleton frame buildings, suspended scaffolds are used when the masonry is laid from the outside. They are supported from above by steel wire cables suspended from outriggers anchored to structural members.

Swing or suspended scaffolds are particularly suitable for winter construction since they are relatively easy to enclose with canvas. Protection is thus afforded to the exterior of the building during the most critical period. Salamanders are placed on the scaffolds and working conditions are very favorable to maintain wall temperatures above freezing day and night. Pole scaffolds on the outside are also easily enclosed. However, for "overhand" work from the inside of the building, the canvas enclosure must be supported by outriggers or supplementary pole scaffolds, and the cost of protection is somewhat greater.

The various types of scaffolds generally used by masons include the following:

1. Horse scaffolds
2. Single pole scaffolds
3. Built-up scaffolds
4. Suspended scaffolds
5. Masons' swinging scaffolds
6. Square scaffolds
7. Tubular pole scaffolds.

(b) Horse scaffolds consist of trestles or horses generally 4 to 5 ft. high and approximately 4 ft. long. The horizontal member or "bearer" is 3 by 4 in. and the legs nominal 2 by 4-in. or 1¼ by 5-in. unfinished stock, all properly braced together with 1-in. boards of the required width. Plank platform or staging supported by the horses are not less than 2 in. nominal thickness and laid with their edges close together.

Horse scaffolds should not be erected more than 3 tiers of horses or more than 12 ft. high. For overhand work from the inside of the building, a single tier of horses is suitable for story heights up to 9-ft. Special "pony" horses approximately 2½ ft. high are sometimes used on the typical single-tier scaffold for story heights up to 12 ft. If only a slight increase in the height of a platform is required, small piers of brick may be built up on top of the scaffold plank to support the raised platform, but not more than 12 in. high. By omitting one or two planks from the platform of the upper or "pony" trestles, a runway is provided for material on the lower tier. Where 2- or 3-tier scaffolds of standard horses are used, supplementary runways are also required in one or two full-tier steps.

Adjustable scaffolding, known as the "Morgen Scaffold," which is designed for wall heights up to 12 ft. has been recently developed. The main members of this scaffold are steel towers, and through a hoisting arrangement which can be operated by one man the platform can be raised so that the brick masons work continuously at waist height. Masonry contractors who have used this scaffold report increase of masonry productivity of approximately 20 per cent with the masons less fatigued at the end of the day than when working on horse scaffolds or other types of non-adjustable scaffolding.

(c) Single-pole scaffolds consist of a single row of posts or uprights set not to exceed 5 ft. from the building and spaced not more than 7 ft. apart. Minimum post sizes are as follows:

Height	Size
Up to 24 ft.	3 by 4-in.
24 to 40 ft.	4 by 4-in.
Over 40 ft.	4 by 6-in. or heavier as required to support all loads with a safety factor of at least 4.

All posts should be thoroughly braced diagonally and connected horizontally with stringer and ledger members. Putlogs, nominal size 4 by 4-in., to carry the plank platform are supported on a ledger at the outer end, and in a wall recess on the inside. At the completion of the wall the recesses are filled in as the scaffolding is removed. Patching of recesses may be eliminated by the use of metal chocks secured to the wall end of the putlogs. They are inserted into the end joints and are supported on the outer 4 in. of masonry.

(d) Built-up or independent pole scaffolds consist of a double row of posts or uprights; the inner row set as near the wall as practicable. Minimum post sizes, allowable heights, and longitudinal spacing are similar to the requirements for single post scaffolds. The platform planking is placed on 2 by 8-in. bearers which in turn are supported on 2 by 8-in. stringers. Diagonal bracing is placed in both directions on the outer row of posts and sometimes on the inner row for very high or heavy duty scaffold.

(e) Suspended scaffolds are generally recommended for all buildings more than 5 stories high constructed of structural steel or reinforced concrete skeleton framing. These scaffolds are supported from above by steel wire cables suspended from overhead steel I-beam outriggers. The complete scaffold consists of supporting frame, steel or wood bearers, plank platform, wood railing, toe-board, wire-mesh protection, and plank overhead covering in addition to the hoisting equipment.

Suspended scaffolds are considered the safest and most economical type for laying masonry from the outside of the building. The hoisting machines permit the masons to work waist-high constantly—the least tiring and most efficient level.

(f) Masons' swinging scaffolds consist of a platform at least 30 in. wide, suspended with steel wire cables from overhead steel beam outriggers. They are often used for painting or cleaning down new or existing walls. The platform may be raised or lowered by means of a windlass to suit the required working position. Wrought iron hangers or stirrups which support the platform at each end should be so designed as to receive the guard-rail and toeboards. The space between the guard-rail and platform is generally filled with a wire netting screen.

(g) Square scaffolds consist of framed wood squares or jacks used in supporting a plank platform in a manner similar to horses or trestle scaffolds. The squares are generally framed from 2 by 4-in. material and are not larger than 5 ft. on each side. Corner bracing is provided on both sides and also 1 by 8-in. diagonal bracing from the center of each frame member to the center of the adjacent member. When the squares are placed in position they are braced laterally by 1 by 6-in. diagonal bracing on both the front and rear sides of the scaffolds.

(h) Tubular pole scaffolds are recommended for exterior or interior work for heights up to 200 ft. All scaffold support members are made from galvanized steel tubing fastened together with steel couples or other approved locking devices. Various members may be described as follows:

Posts are vertical supporting members.
Runners are lengthwise horizontal members.
Bearers are supports for the plank platform.
Braces are diagonal members applied lengthwise and crosswise as required.

These members are available in various weights depending on the height and maximum platform load. Post sizes for scaffolds used in masonry work are generally 2 or 2½ in. o.d. Whenever possible, the scaffold should be tied to the building through the window openings. This will greatly reduce the necessary amount of crosswise bracing.

(i) Towers. Material-hoist towers erected outside of buildings are usually constructed of timber or tubular steel sections.

Post sizes for wooden towers will vary from 4 by 4 in. for the top section to 6 by 8 in. for the lower portions as determined by height and cage capacity. Horizontal ties of 1 by 6-in. to 2 by 8-in. material are required at 6- or 8-ft. maximum intervals, with diagonal cross bracing between horizontal ties on all four sides except at loading or unloading platforms. At these openings, additional corner bracing is generally required. Guide rails for cages should be of sound lumber free from knots or other defects and must be kept in perfect alignment at all times.

All hoist towers should be securely anchored to the building or guyed to well buried "dead-men" of adequate size.

The overhead framework of all towers must be of sufficient strength to carry the entire dead and live loads with a safety factor of 5.

Steel tubular towers are extensively used by contractors for hoisting all types of building material. They are easily erected and dismantled and, due to their strength and stability as well as economy, have practically supplanted wooden towers in modern construction. Towers up to 200 ft. in height may be constructed with 2½-in. o.d. tubing. This compares with 6 by 6-in. or 6 by 8-in. timbers required for the lower half of 200-ft. maximum height wooden towers. Heavy duty tubular steel towers consisting of 3-in. o.d. tubing are used for heights up to 850 ft.

811. STUCCO AND PLASTER

(a) Exterior Stucco Finish. Brick or tile provide an excellent base for the direct application of portland cement stucco as well as lime and gypsum plaster. In addition to the positive bond between the materials, the walls are rigid and free from movement due to changes in moisture and temperature conditions. This is an important characteristic of clay masonry construction.

Units intended for use with stucco finish are available in all standard wall thicknesses for single or multiple unit construction. Although dove-tailed or grooved finishes are the most common, approved surfaces may be smooth, scored, combed, or roughened in accordance with ASTM specification requirements for load-bearing and non-load-bearing wall tile. Tests conducted at the National Bureau of Standards on the adhesion of plaster to the various surface finishes show excellent bond results for all types.

In order to be assured of a permanent stucco finish, the essentials of building construction as required for all kinds of materials must be followed. Footings should be solid and unyielding, and foundation walls should be constructed of proven materials, capable of carrying the sustained loads.

Stucco is one of the oldest and most versatile types of exterior wall finish. Examples exist today which have endured for centuries, particularly in the Central European countries. Investigation in these countries indicate, however, that less dense mixtures are used than in this country and surface scraping is often resorted to in order to remove the rich and fatty surface film. It is noted that the smooth steel trowel or even the wood-float finish may be responsible for surface crazing. These hair cracks are due to shrinkage of the rich surface material and are more apt to occur when a wet mix is used. To prevent crazing, a felt polisher, cork or carpet covered float may be used, which produces a reasonably smooth surface.

1. Recommended Mixes. Mortar for exterior cement stucco or cement plastering should consist of the following proportions, in accordance with present (1953) recommendations: One sack of cement to 3 cu. ft. of sand, and approximately 10 lb. of hydrated lime as required for workability.

Dry hydrated lime is added to the cement and thoroughly mixed dry, before spreading over the prepared sand bed. These ingredients are mixed until a uniform color is obtained, approximately ⅔ of the required water is then added, and the remainder as required to any dry spots until a uniform mass of the proper consistency is obtained.

2. Application. All hangers, fasteners, trim, or other fixed supports or projections should be in place previous to the application of any stucco. Flashings must be provided above and below all openings, at parapets, spandrels, and at all other points where water may enter.

Stucco is generally applied in two coats on brick or structural tile walls for a total thickness of not more than ⅝ to ¾ in. Generally the surface must be evenly wetted to prevent a rapid loss of moisture from the mortar. Due to the range in absorption of various clay products, experience of the skilled craftsman will best determine the amount of wetting required. In any event the walls should not be saturated and in colder and damp weather only a light application may be required. On hard, low absorption units no wetting may be necessary.

The first coat should be well troweled to insure proper mechanical bond. It should be doubled back to bring the plaster to the required thickness and brought to a true and even surface by the use of a straight edge. Before the coat has set it should be cross-hatched to insure a strong mechanical key for the finish coat. Plastering should be carried on continuously without allowing the plaster to dry at the edges.

The second or finish coat should be applied not less than seven days after the first coat, and should be from ⅛ to ¼ in. in thickness depending on the type or roughness of final surface texture. If desired, white cement may be

substituted for the final coat in place of ordinary portland cement and it may be tinted by adding mineral color pigments to the mixture. Because of the difficulty of obtaining a uniform color when job-mixed, a prepared exterior stucco finish is often used for the finish coat. It is available in white and various popular shades and is especially designed to resist moisture and temperature changes.

3. Curing. After the first coat is applied, it should be kept damp for at least two days and allowed to dry gradually. Before the next coat is applied it must be evenly wetted, but not saturated. This is done by throwing the water from a large brush as the work progresses. The skilled plasterer will use just the right amount of water to obtain the proper workability and maximum adhesion.

The finish coat should also be kept damp for at least two days either by sprinkling or covering with wet burlap or similar material.

Unless adequate precautions are taken to insure proper curing temperature, stucco should not be applied when the temperature is below 40° F.

The following recommendations on the pneumatic application of stucco are quoted from the 1940 Report of the Joint Committee on Standard Specifications for Concrete and Reinforced Concrete:

"(a) Construction utilizing pneumatically applied mortar has been in the hands of a few organizations specializing in this field and utilizing equipment produced by a very limited number of manufacturers. This type of construction has been in use for many years and considerable advance in technique and equipment has been made during that period, but there is still considerable divergence in procedure between the different users. The purpose of this section is to provide a resume of the accepted practice as supplementary to the general specifications.

"(b) The results obtained in pneumatically applying mortar depend to a large extent on the skill of the operator and the work should be done only by experienced men.

"(c) Two types of equipment are available, single or double chamber. The single chamber type is operated intermittently, and is adaptable to small jobs only. The double chamber type which can be operated continuously is the most common type in use.

"(d) The grading of the sand is important in obtaining high strength and density. The sand should be passed through a ⅜-in. sieve and have a fineness modulus of 3.0 or less. Stone sand subject to the same grading limitations as natural sand gives satisfactory results.

"(e) The proportions of cement to sand for various purposes should range from 1 part cement to 3 parts sand, to 1 part cement to 4½ parts sand, in terms of dry and loose volumes. The mortar in place will have a somewhat lower proportion of sand because of the loss due to rebound.

"(f) Control of the water content is important for pneumatically placed mortar in order to obtain proper placement. The water content should be such that a very slight film of water will form on the surface of the applied material. Insufficient water results in dry porous spots and excess water causes low strength and slipping of the mortar.

"(g) The air and water pressure should be maintained uniform. The water pressure should be at least 15 psi higher than the air pressure which should be sufficient to give proper velocity to the material leaving the nozzle. To avoid excessive impact the air pressure should not exceed 75 psi.

"(h) The sand should be neither excessively dry nor wet—about 4 per cent moisture by weight is desirable. Sand and cement should be thoroughly premixed by machine as very little mixing takes place during the application.

"(i) The velocity of the material that leaves the nozzle should be maintained uniform and should be such as to produce a minimum of rebound of the sand. The nozzle should be held between 3 and 4 ft. from the surface being covered and should be kept moving to obtain a uniform coating.

"(j) Special care should be taken in the removal of forms and in the curing to avoid cracking of the thin sections. Placing at temperatures below 50° should not be attempted unless adequate facilities are available for keeping the mortar above 50° during and after placing. The surface to which mortar is applied should be free from frost.

"(k) Shooting strips should be used at corners, edges, and on surfaces where necessary to obtain true lines and proper thickness. The surface of the mortar may be 'steel floated' when a very smooth finish is desired.

"(l) The placement equipment should be thoroughly cleaned whenever work is to be stopped for a period in excess of 30 min.

"(m) The position and size of reinforcement where required and the thickness of coating varies with the use of the pneumatically applied mortar and should be within the limitations given below."

(Only the limitations affecting stucco on tile, terra cotta, brick and concrete are quoted.)

Use	Reinforcement	Thickness
Stucco or Facing on Tile, Terra Cotta, Brick, and Concrete.	Not necessary except for badly disintegrated surfaces in which case use galvanized welded fabric equal to 0.2 per cent of cross-sectional area of the mortar securely attached to the structure and located at middle of coating.	⅝ in. to 1 in. placed in two layers, second ⅛ in. thick. Screed to proper surface before applying second layer.

(b) Interior Plaster Finish. Most authorities recommend the use of furred plaster on the interior surfaces of exterior walls, particularly for 8-in. masonry wall construction in northern climates. The furring may be wood or metal strips fastened to the wall. Lath is then attached to the strips to provide the plaster base or key. It may be wood or expanded metal, plasterboard, or rigid insulation. The furring space increases the insulation value of the wall and prevents possible dampness due to condensation as well as from leakage or capillarity.

If desired, dampproofings consisting of troweled or sprayed asphalt preparations may be used on the inside surfaces of exterior walls when the plaster is applied to the masonry. This is intended to protect the plaster from contact with moisture which may penetrate the walls by capillary action. These dampproofings do not in any way improve the bond of plaster to the wall but do have sufficient adhesion to provide very satisfactory performance.

1. Types of Plaster. For ordinary plaster, the cementitious material is usually gypsum or lime. Portland cement and Keene's cement, a hard-burned gypsum product, are also used, and often required in certain interior areas. Sand is the principal aggregate with the addition of hair or wood fiber in some cases, particularly for scratch coats. These materials are mixed with water to develop the plasticity of the cementitious material and to react chemically in the hardening process.

Lightweight aggregate, such as Zonolite or Perlite, is being used in increasingly greater volume to replace the sand used in ordinary plaster. These materials provide greater insulation, increased fire resistance and sound absorption. In addition, they import a high degree of resistance to cracking due to nailing, shock, or impact. Manufacturers' directions should be carefully followed regarding proportions, materials and methods of application.

2. Recommended Mixes. Gypsum or hardwall plaster is mixed 1 part gypsum to 2 parts sand by weight for the scratch coat on three-coat work, and 1 part gypsum to 3 parts sand for the brown coat or the base coat for two-coat work.

Wood Fiber Gypsum Plaster is mixed with water only for the scratch coat on lath and mixed 1 part fibered gypsum to 1 part sand, by weight, for the brown coat or the base coat on masonry.

Lime Plaster for scratch coat on three-coat work consists of 1 volume of putty and not more than 2½ volumes of sand gaged with 10 lb. of Keene's cement per cu. ft. of putty, with sufficient hair or wood-fiber to form a binder. For the brown coat or the base coat of two-coat plaster applications on masonry, 3 volumes of sand are permitted and the percentage of fiber or hair reduced ⅓ to ½. Where no finish coat is desired the fiber is omitted from the brown coat.

Portland Cement Plaster is mixed and applied as described for exterior stucco finish.

3. Finish Coats. The finish coats for gypsum and lime plaster should be a prepared hard finish applied in accordance with the manufacturers' directions, or a mixture of lime putty and calcined gypsum, or Keene's cement. It generally consists of 1 part calcined gypsum (plaster of paris), or of Keene's cement, to 3 parts lime putty by volume. The lime putty should be a stiff mixture of hydrated lime or pulverized quicklime and water. It should be completely slaked and allowed to cool before using. Putty should be allowed to soak in accordance with lime manufacturers' instructions, and must be kept moist until used.

Keene's cement hard finish is mixed in the proportions of 3 parts Keene's cement to 1 part of lime putty, by volume.

4. Application. When plaster is applied directly to the surface of clay masonry walls and partitions, only two-coat work is required. The brown or base coat may be doubled back by applying a thin coat to fill the small depressions and then brought out to the grounds and straightened to a true surface. If a finish coat is used, the base course must be carefully scored to create a key for adhesion. The finishing coat should be applied after the undercoat has become about dry.

For three-coat work, as required on lath, the scratch coat should be applied with sufficient material and pressure to provide a proper key, and then scratched vertically and horizontally to obtain a rough surface for the brown coat. Where lime plaster is used, the brown coat should be applied only after the scratch coat has thoroughly dried. Where gypsum plaster is used, the brown coat should be applied only after the scratch coat has become thoroughly hard.

Keene's cement lime putty finish and the Keene's cement hard finish should be applied over a thoroughly set base coat which is nearly, but not quite, dry. If the base coat has dried out it should be lightly sprayed with water, but not soaked, before the finishing coat is applied.

In general, the finishing coats should be scratched in thoroughly, laid on well, doubled back, and filled out to an even surface. These coatings are allowed to dry for a few minutes and then troweled and brushed to a smooth finish. The total thickness of the finishing coat should be not less than 1/16 in., nor more than ⅛ in.

812. PAINTING

(a) Exterior. Since the natural colors, shades and textures of brick and tile units are both pleasing and permanent, painting is not required on walls constructed of these units except to obtain a special decorative appearance or architectural treatment. Furthermore, paint should not be used as a waterproofing medium unless recommended to seal shrinkage and separation cracks in the mortar.

Conclusive tests show that very little dampness, even under severe exposure conditions, will penetrate the clay masonry units. However, incomplete filling of mortar joints or the use of unwetted units particularly in warm weather may provide a path for the penetration of moisture under certain exposure conditions.

Except for the specific purpose described above, painting of structural clay masonry walls is primarily a question of taste to be decided by the owner himself. No one should be misled, however, into believing that painting of clay masonry walls adds to the structural qualities of a building. The cost of painting must be charged to appearance alone and cannot be offset by increased permanence or resistance to weather as in the case of frame structures.

Claims made that painted walls are superior to unpainted ones from the standpoint of insulation and heat transmission are too theoretical to warrant serious consideration. It is true that saturated materials have higher coefficients of conductivity than dry materials. However, clay masonry units are rarely, if ever, saturated in a wall even under the most adverse conditions and heat transmission is relatively unimportant during or immediately following heavy rains. In extremely cold weather or during very hot weather, clay masonry walls are usually dry.

Increases in reflected light and reduction of exterior exposed wall surface temperatures obtained by the use of white paint on red masonry surfaces can be easily verified by test. However, painted masonry walls are difficult to clean and the effectiveness of the painted surface (particularly exterior walls) decreases rapidly with weathering and the accumulation of soot and dirt. A permanent treatment and one less expensive to maintain would be the use of glazed tile or light-colored facing units.

Exterior coatings on masonry walls which have a high resistance or are impermeable to vapor travel will cause condensation within the walls when the wall temperature falls below the dewpoint. This condensation will tend to saturate the wall and may cause leakage at the interior surfaces. A highly vapor-resistant coating will also prevent the rapid drying of masonry walls which, in the literature on the subject, is frequently referred to as "breathing". Brick and tile walls, particularly when built of high absorption units, dry rapidly following periods of partial saturation due to capillary action which draws the water entrapped in the wall to the face of the units. If this breathing action is retarded or eliminated, water which may enter the wall through openings in the mortar joints will accumulate until the wall approaches saturation. This condition, not only contributes to dampness on interior surfaces but, if the wall is subjected to freezing in a saturated or near saturated

state, the disrupting effects of freezing are many times greater than would be the case if the wall were relatively dry.

This is the reason that many brick, which have resisted freezing and thawing for years when exposed in exterior walls, have spalled after one or two years' exposure when painted.

In general, if brick are to be painted, they should have a higher resistance to freezing and thawing than if unpainted, and paints having low resistance to vapor travel are to be preferred.

1. Cement Paint. In addition to the breathing property of cement paint, another reason for a preference over oil paint is that the cement paint is applied to a damp masonry wall while oil paints can be applied only to dry walls.

The following are the recommendations of the Portland Cement Association for the application of cement paint to masonry walls:

"Portland cement paint should be carefully applied in accordance with manufacturers' directions. Surfaces should be damp when cement paint is applied so that even absorption is obtained. High winds, excessive heat and strong sunshine will dry cement paint quickly and render it ineffective as a waterproofing agent unless proper precautions are taken to keep it damp until properly cured. Cement paint should be cured as carefully as concrete. It should be kept damp—not allowed to dry out—until it has set thoroughly. After the first coat has hardened sufficiently to prevent injury to the surface, it should again be wetted down just before applying the second coat. The second coat should also be kept damp until it has thoroughly cured and hardened—preferably for at least 48 hr. after application."

Portland cement paints should not be used over old coats of oil paint, whitewash or casein paints.

2. Oil Paints. Before oil paints are applied to new masonry, the walls should be thoroughly dried out and cleaned of all dirt and mortar particles. If oil paint is desired for any reason on new masonry within 3 months after construction, it is usually necessary to apply a wash coat of zinc sulphate solution (2 lb. to 1 gal. of water) to neutralize the alkali in mortar joints. This treatment, however, is not always completely effective, particularly if moisture finds its way into the wall. In any case the paint manufacturers' directions should be carefully followed since some oil paints can be applied over a zinc sulphate wash while others cannot.

If there is any efflorescence on the wall, it should be removed by brushing while dry. The wall should then be washed with clean water and then washed with a 10 per cent solution of muriatic acid, after which it should be washed again with clear water.

In order to secure satisfactory results, the masonry should receive three coats of paint—a priming coat or first coat, a body or second coat and a finishing or third coat. At least four or five days should be allowed between coats. A typical primer coat consists of exterior spar varnish containing an equivalent volume of house paint or color in oil.

Many paints are produced expressly for masonry surfaces and the application, in any event, should be in accordance with the manufacturers' directions.

A word of caution, however, is justified regarding the claims for highly advertised so called "plastic paints". Tests conducted by the National Paint, Varnish and Lacquer Association and reported in Circular No. 701, indicate that "most of them appear to be no better than, and, in some instances, not so good as regular moderately priced trade sales items."

(b) Interior Paint Finish. Painted finishes are sometimes specified for the interior masonry surface of exterior walls and for unplastered masonry partitions. Where decorative finishes of this type are desired, the natural smooth surface of clay masonry is particularly suitable for painting. To obtain best results, the surface porosity must be taken into consideration by the use of special recommended priming coats, particularly when using oil-base paints. Manufacturers' directions should be carefully followed as required for the various types of paint, number of coats and methods of application.

If oil paint is used on new masonry construction (less than 3 months old), the walls should be neutralized with a zinc sulphate solution as described above for exterior surfaces.

Cement-base paints are satisfactory for interior masonry surfaces, also casein paints when used on dry interior walls.

813. REPAIR AND MAINTENANCE

(a) General. Properly constructed clay masonry walls are remarkably free of costly repair and maintenance. When required, the repair of existing masonry is often more difficult and costly than proper construction of new work. Care in the selection and use of mortar, wetting brick, adequate flashing and tooling of joints will add only a small amount to the initial cost but will insure a low maintenance overhead throughout the life of the structure.

(b) Tuck-pointing. Where the mortar joints have softened or distintegrated or large cracks are noted, it will be readily apparent that protective measures must be taken to correct or prevent leaky walls or structural weakness.

This is done by cutting out all loose or disintegrated material for a depth of at least ½ in. and repointing or filling with proper mortar. If the work is being done to correct leakage, all joints should be cut out in the affected area as it may be very difficult in some cases to determine the defective joints by visual inspection. If no leakage has been noted and the repointing is being done as maintenance work, it is necessary to remove only the defective mortar.

When the cutting is completed all dust and loose material must be removed by brushing or, preferably, with a water jet. If water is used in cleaning the dust from the joints, no additional wetting may be required. The repointing should not follow immediately after the joints are washed and little if any wetting will be necessary when the walls are constructed of low absorption units.

Mortar for tuck-pointing should never be denser than the original mortar and preferably pre-hydrated by mixing 1 hr. to 2 hr. before using, with only a portion of the required mixing water sufficient to produce a damp mass of such consistency that it will retain its form when pressed into a ball with the hands, but will not flow under the trowel. The mortar should then be allowed to stand for a period of not less than 1 hr. nor more than 2 hr., after which it is thoroughly remixed with the addition of sufficient water to produce satisfactory workability. This greatly improves the workability and much of the initial shrinkage is eliminated. The mortar should then be packed in tightly in thin layers and finally tooled to a smooth compact concave surface.

(c) Waterproofing. When the mortar cracks and openings are small, a two-coat application of cement-sand grout brushed vigorously into the mortar joints will provide an effective waterproofing method. A typical recommended mixture by volume consists of 1 part portland cement, ¼ part limestone flour, powdered flint or hydrated lime and 1 part sand passing a No. 30 sieve. The joints should be thoroughly wetted before applying the grout.

(d) Flashing. The omission of flashing, the use of an improper type, or good flashing incorrectly placed is often responsible for serious masonry problems. Unflashed brick and cast or cut-stone sills, projecting courses, and particularly copings, generally result in leakage or, at least, disfigured walls caused by efflorescence. The only proper solution is an expensive repair job which requires removing the brick or stone and placing suitable flashing around and under it. When continuous flashing is required in existing walls at spandrels or other locations, it can be placed by removing alternate masonry sections in widths up to 2 or 3 ft. After the flashing is placed and masonry properly aged, the intermediate pier sections can be removed and the flashing completed.

(e) Caulking. Improper caulking is often responsible for water leakage around door and window frames. The omission of caulking is easily corrected by filling all cracks with a good elastic caulking compound by means of a pressure gun. On the other hand, if the original caulking has cracked, peeled or separated, it should be removed and replaced with new compound. Unless proper pressure is used, only a thin film of caulking compound will be placed. Even with good material, this will soon become ineffective. Thin films should be removed and properly replaced before serious damage results.

APPENDIX A

BUILDING CODE LATERAL FORCE REQUIREMENTS

The California State Health and Safety Code requires, with very few exceptions, that all buildings be designed to resist a lateral force based upon a factor of $C = 0.02$, or a specified wind pressure, whichever may be the greater. Most cities on the Pacific Coast have earthquake requirements which are considerably greater than this 2 per cent factor. The Uniform Building Code, 1952 edition, contains the following earthquake requirements as a part of its appendix:

Note: Communities which are under the Uniform Building Code but are not in an active seismic area generally do not adopt the earthquake requirements.

"*Horizontal Force Formula.* In determining the horizontal force to be resisted, the following formula shall be used:

$$F = CW$$

where F = the horizontal force in pounds
W = the total dead load,

tributary to the point under consideration, except for warehouses and tanks, in which case W shall equal the total dead load plus the total vertical designed live load tributary to the point under consideration. Machinery or other fixed concentrated loads shall be considered as part of the dead load."

C equals a numerical constant as shown in Table A-1 (see page 240).

It is to be noted that the map referred to in Table A-1 places all of California, except its Central Valley area and the Puget Sound area of Washington, in Zone 3. The balance of the Western States, except for parts of Oregon, Nevada, Arizona and Montana, are in Zone 2. (Approximate summary—see map in Uniform Building Code, 1952 edition.)

Los Angeles County earthquake requirements are essentially as given by the Uniform Building Code.

The City of San Francisco has a building code, adopted in 1948, which contains the following requirements regarding earthquake design forces:

"(a) *Minimum Lateral Load.* Every building or other structure shall be designed and constructed to withstand a minimum seismic lateral force, acting from any horizontal direction, as given by the following formula:

$$F = CW$$

in which F = horizontal lateral seismic force at the elevation considered,
C = a numerical coefficient as given hereafter,
W = the total dead and live column design load at and above the plane under consideration. The design loads shall be as given in Section 2210 as modified by Section 2214.

TABLE A-1*

HORIZONTAL FORCE FACTORS

Part or Portion	Value of C ①	Direction of Force
Floors, roofs, columns and bracing in any story of a building or the structure as a whole ②	$\frac{.15}{N③ + 4\frac{1}{2}}$	Any direction horizontally
Bearing walls, non-bearing walls, partitions, free standing masonry walls over 6 ft. in height	.05 With a minimum of 5 lb. per sq. ft.	Normal to surface of wall
Cantilever parapet and other cantilever walls, except retaining walls	.25	Normal to surface of wall
Exterior and interior ornamentations and appendages	.25	Any direction horizontally
When connected to or a part of a building: towers, tanks, towers and tanks plus contents, chimneys, smokestacks and penthouses	.05	Any direction horizontally
Tanks, elevated tanks, smokestacks, standpipes and similar structures not supported by a building	.025	Any direction horizontally

* Table 23-C, Uniform Building Code

① See map for zones. The values given C are minimum and should be adopted in locations not subject to frequent seismic disturbances as shown in Zone 1. For locations in Zone 2, C shall be doubled. For locations in Zone 3, C should be multiplied by 4.

② Where wind load as set forth in Section 2307 would produce higher stresses, this load shall be used in lieu of the factor shown.

③ N is number of stories above the story under consideration, provided that for floors or horizontal bracing, N shall be only the number of stories contributing loads.

"(b) *The Coefficient C.* The values of the coefficient C to be used in the design of structures are expressed in the following formula and Table A-2.

$$C = \frac{2}{24 + N}$$

in which N = the number of stories counting from the top of the building to the plane under consideration. The top story of the building shall be designated as number one.

"(c) *The Coefficient C Applied to Parts of Structure.* In order to determine the required strength of parts of the structure, the coefficient C shall be as follows:

Exterior walls	.10	with a minimum of 15 lb. per sq. ft.	Normal to surface of wall
Interior walls, partitions	.10	with a minimum of 5 lb. per sq. ft.	Normal to surface of wall

Parapet walls, exterior and interior ornamentations and appendages	.50	Any direction horizontally
Towers, tanks with and without contents, chimneys, smokestacks and penthouses when connected to a part of a building	.20	Any direction horizontally

TABLE A-2*

SEISMIC COEFFICIENTS C

Stories Numbered From Roof	Values of C	Modifications
1	.080	1. Building on marshy or filled ground. Buildings on marshy or filled ground, whether carried on piles or not, shall be designed for values of C 25 per cent higher than the tabulated values opposite, unless designed with a continuous reinforced concrete foundation slab or grillage capable of uniformly distributing the foundation loads in which case the usual values of C shall govern.
2	.077	
3	.074	
4	.071	
5	.069	
6	.066	
7	.064	
8	.062	
9	.060	2. Buildings on rock. For buildings on a foundation of rock extending throughout the entire area of the structure, 15 per cent reduction in the values of C is permissible.
10	.058	
11	.057	
12	.055	
13	.054	
14	.053	3. Theaters. Values of C for all stories of theaters shall be taken as .10.
15	.051	
16	.050	
17	.049	
18	.048	
19	.047	
20	.046	
21	.045	
22	.044	
23	.043	
25	.041	
26	.040	
27	.0393	
28	.038	
29	.038	
30	.037	

* Table 23-a, Building Code, City of San Francisco.

The California State Administrative Code under the "Field Bill" requires that all school buildings in the state be designed to resist lateral forces. The Division of Architecture of the Department of Public Works of California which initiates and enforces regulations to accomplish this has established the following requirements regarding the lateral forces to be used in earthquake design:

"304. *Amount of Earthquake Force.* The amount of the earthquake force shall be considered to be applied in any direction and shall be not less than that given by the formula:

$$F = CW_{D+L}$$

in which F = force of earthquake in pounds

C = a coefficient as hereinafter specified

W_{D+L} = total dead weight of building plus total live load. (Except that said live loads may be reduced as provided in Section 205.)

"305. *Values of Coefficient C in Formula of Section 304.*

TABLE A-3

VALUES OF COEFFICIENT C

Safe Resistance of Foundation Soil to Vertical Loads	Coefficient C
4 tons per sq. ft. or over	.06
More than 2 but not less than 4 tons per sq. ft.	.08
2 tons per sq. ft. or less	.10

"When it is proposed to use reduced seismic factors on the basis of allowable soil pressures in excess of 2 tons per sq. ft., the allowable soil pressure shall be determined by acceptable tests made by a recognized soil technologist who shall make arrangements with the Division of Architecture regarding inspection and supervision prior to beginning such tests.

"306. *Allowable Reduction of Bending Stresses by Vertical Load.* In calculating maximum tensile fiber stresses due to wind forces, it is permissible to deduct the direct dead-load compression due to gravity from the tension due to bending. However, when considering seismic forces, the maximum tensile fiber stresses may be reduced by not more than 75 per cent of the direct stress due to vertical dead loads.

"307. *Seismic Force for Special Structures.* Tank towers, tanks, chimneys, smokestacks, and marquees attached to a building shall be designed to resist a lateral force of 20 per cent of the dead and live loads as set forth in these rules and regulations. Parapet walls, cantilever walls above roofs, exterior ornamentation and appendages shall be designed to resist a lateral force of 100 per cent of their weight. The structural members of the building supporting the special structures named above need only be designed to resist a lateral force based on the value of coefficient C applicable to the building in general.

"308. *Value of Coefficient C for Buildings Supported by Pile Foundations.* For buildings supported on piling, the coefficient C shall be the same as that for a soil having a resistance not greater than 2 tons.

"309. *Value of Coefficient C for Buildings Supported on Foundation Piers Extending to Firm Soil or Rock.* For buildings supported on reinforced concrete foundation piers (as outlined in Section 1110) extending to firm soil or rock, the building shall, for the purpose of determining C, be assumed to be carried on footings supported by a soil having a resistance to vertical load of 3 tons per sq. ft.

"310. *Required Resistance Against Horizontal Torsional Moments.* The vertical structural units of the building which resist the force of the earthquake shall be so arranged that, in any horizontal plane, the centroid of such resisting structural units is coincident with the center of gravity of the weight of the building, or else proper provision shall be made for the resulting torsional moment on the building.

"311. *Distribution of Horizontal Shears.* The total horizontal shear at any level shall be distributed to the various resisting units at that level in proportion to their rigidities, giving due consideration to the distortion of the horizontal distributing elements.

"312. *Allowable Use of Permanent Structural Elements for Resistance to Earthquake.* Reinforced concrete or masonry walls with all permanent structural elements capable of providing resistance shall be assumed to act integrally with structural frames in resisting the shears and moments due to the horizontal forces unless specifically designed and constructed to act independently from said structural frames."

The Pacific Fire Rating Bureau recommends the following lateral forces:

"*Horizontal Force Formula.* The horizontal force to be resisted shall be determined as follows:

$$F = CW$$

where F = the horizontal force in pounds

W = the total dead load, plus 25 per cent of the vertical designed live load at and above plane or elevation under consideration, except for buildings one story in height, and warehouses and similarly loaded buildings where W shall equal the total dead load plus 50 per cent of the total vertical designed live load at and above the plane or elevation under consideration. (In calculating W for one-story buildings, the live load on the roof shall in no case be assumed as less than 15 lb. per sq. ft.) All fixed, permanent contents, such as machinery, desks, etc., shall be considered part of the dead load.

C = a numerical constant and shall be as specified in Table A-4 (Page 244).

TABLE A-4

Building or Structure and Part or Portion Thereof	Value of C	Direction of Force
1. The building or structure as a whole	.10	As specified above
2. Bearing walls, curtain walls, enclosure walls, fire division walls, panel walls	.20	Normal to surface of wall
3. Fire wall, parapet wall, cantilever parapet and cantilever walls	1.00	Normal to surface of wall
4. Exterior and interior ornamentations and appendages, marquees	1.00	Any horizontal direction
5. Chimneys, smokestacks, flag poles, penthouses	.20	Any horizontal direction
6. Tanks, towers, tanks and towers, plus contents		See special rules governing design

"Where the building or structure rests on piling or filled ground, or on ground which will withstand safely a vertical load not greater than 1 ton per sq. ft., the value of C in Items 1, 2 and 5 above shall be increased at least 25 per cent."

APPENDIX B

DESIGN OF REINFORCED BRICK BEAMS AND SLABS

B-1. DISCUSSION OF DESIGN THEORY

Tests indicate the structural performance of reinforced brick masonry is analogous to that of reinforced concrete and that the formulae used in the calculation of stresses and deflections in reinforced concrete flexural members can be used in calculations for similar reinforced brick members. Also, as indicated in Chapter 2, both types of construction perform like homogeneous beams in that all relations of load and moment to deflection and stress are linear over ranges of loading well past design loads.

Similar assumptions have been made in deriving the flexure formulae for reinforced brick masonry to those made for reinforced concrete. Most text books on reinforced concrete design contain complete discussions of these assumptions and theory of design. Only an abridged derivation of the flexure formulae will be given here.

B-2. NOMENCLATURE

The following notations are used in the derivation of flexure formulae for working loads:

- A_s Effective cross-sectional area of tension reinforcement in a beam or slab
- b Breadth of rectangular beam or slab
- d Depth from compression face of beam or slab to center of gravity of longitudinal tension reinforcement
- E_s Modulus of elasticity of steel
- E_m Modulus of elasticity of masonry
- n E_s/E_m = ratio of modulus of elasticity of steel to that of masonry
- f_m Compressive unit stress in extreme fiber of masonry
- f_s Tensile unit stress in longitudinal reinforcement
- p Ratio of effective area of tension reinforcement to the effective area of masonry in a beam or slab = A_s/bd
- j Ratio of lever arm of resisting couple to depth d
- k Ratio of depth of neutral axis to depth d (from compressive face of beam or slab)
- M Bending or resisting moment in general
- M_s Resisting moment as determined by steel
- M_m Resisting moment as determined by masonry
- T Total tension in steel at any given vertical section of a beam
- C Total compression in masonry at any given vertical section of a beam
- e_s Unit deformation of steel due to f_s
- e_m Unit deformation of masonry due to f_m
- z Depth from compression surface of beam or slab to resultant of compressive stresses.

B-3. FLEXURE FORMULAE FOR WORKING LOADS

The straight-line theory of stress distribution for reinforced brick masonry beams is based on the following assumptions: The unit stress in the steel is within the elastic limit. Plane sections before bending are assumed to remain plane sections after bending. The unit stress deformations in the masonry at any given section of the beam are considered to vary as the ordinates to a straight line. These ordinates start with zero and increase directly with the distance from the neutral axis. Tension in the masonry is neglected. Bond between steel and masonry is perfect; i.e., no slipping of steel occurs.

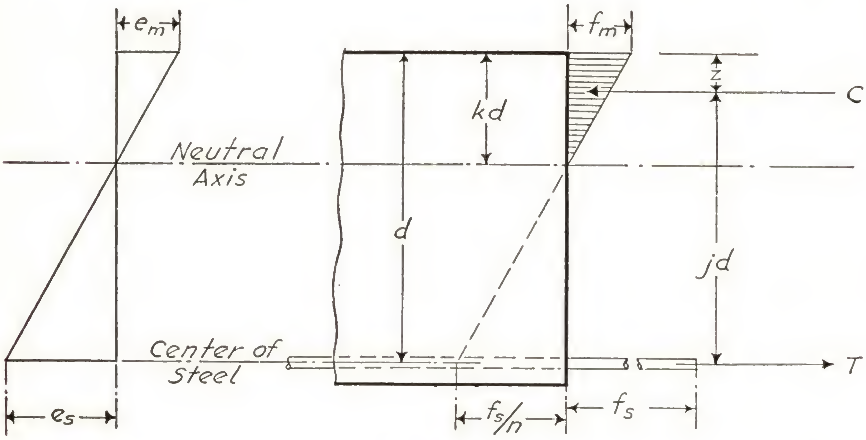

Fig. B-1

Straight-line Stress Distribution

From the geometrical relations shown in Fig. B–1, it may be seen that:

$$\frac{e_s}{e_m} = \frac{d - kd}{kd}$$

Since $e_s = f_s/E_s$, $e_m = f_m/E_m$ and $E_s/E_m = n$, we have:

$$\frac{f_s}{nf_m} = \frac{d - kd}{kd} = \frac{1 - k}{k} \quad \text{(1)}$$

Under normal conditions of design, where the beam is level and all loads and reactions are vertical, the total compression C and the total tension T on a section are opposite and equal; therefore, it follows that:

$$f_sA_s = \tfrac{1}{2} f_m kdb \quad \text{(2)}$$

Eliminating f_s/f_m between equations (1) and (2) and introducing p, gives:

$$k^2 = 2\,pn\,(1 - k) \quad \text{(3)}$$

Solving equation (3) results in:

$$k = \sqrt{2pn + (pn)^2} - pn \quad \text{(4)}$$

Equation (4) is basic and shows that the proportionate distance k which fixes the position of the neutral axis is the same for all beams having the same percentage of steel reinforcement and when built in the same manner of brick and mortar having like physical characteristics.

Since the form of the compressive area shown in Fig. B-1 is that of a triangle, it follows that the distance z to the compressive resultant C is situated one-third of the distance down from the compressive face of the beam to the neutral axis; therefore, the arm of the resisting couple C and T is:

$$j = 1 - k/3 \quad \ldots\ldots (5)$$

As the value of k increases, the value of j decreases, but not in the same ratio, as may be seen by reference to Tables B-2 to B-7 which show that k and j vary according to the values of p and n. It will be observed that j varies only slightly with p, and that for a value of n equal to 15, and with a variation in the value of p between the common design limits of .0075 and .01, the average value of j is approximately $\frac{7}{8}$. This fraction is commonly used by many designers for approximate designs and for checking.

Values of k and j for percentages of reinforcement ranging from p = .003 to .017 and for various values of n have been plotted on Fig. B-2.

If a beam is under-reinforced, its resistance to bending is limited by the allowable tensile stress in the steel reinforcement and its moment of resistance is:

$M_s = Tjd = A_s f_s jd = f_s pjbd^2$, from which is obtained:

$$f_s = \frac{M}{A_s jd} = \frac{M}{pjbd^2} \quad \ldots\ldots (6)$$

If a beam is over-reinforced, its resistance to bending is limited by the crushing strength of the brickwork and its moment of resistance is:

$M_m = Cjd = \frac{1}{2} f_m kdbjd = \frac{1}{2} f_m kjbd^2$, from which is obtained:

$$f_m = \frac{2M}{jkbd^2} = \frac{2pf_s}{k} \quad \ldots\ldots (7)$$

For approximate calculations and for checking, the average values of $j = \frac{7}{8}$ and $k = \frac{3}{8}$ may be used. Equations (6) and (7) then become:

$$M_s = \tfrac{7}{8} A_s f_s d \quad \ldots\ldots (6a), \text{ and}$$
$$M_m = \tfrac{1}{6} f_m bd^2 \quad \ldots\ldots (7a)$$

Which of the two resisting moments, M_s or M_m, is the smaller must be determined in each specific case. A comparison of the values of $f_s p$ and $\frac{1}{2} f_m k$ is sufficient for such determination.

To design a beam or slab having equal resisting moment in tension and compression at working unit stresses, the value of k is eliminated between equations (1) and (7) and an equation is derived for p. This value is:

$$p = \frac{\frac{1}{2}}{f_s/f_m\,(f_s/nf_m + 1)} \quad \ldots\ldots (8)$$

The value of p for different values of f_s and f_m and for values of n = 10, 12, 15, 18, 20 and 25 are shown in Tables B-2 to B-7.

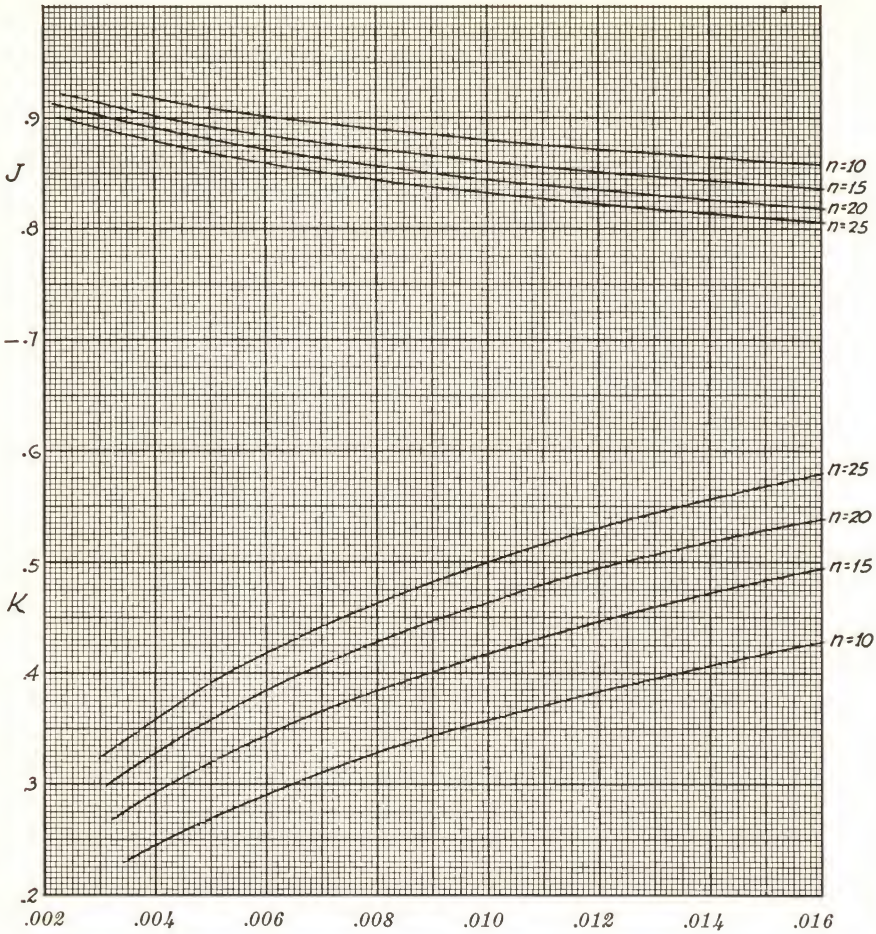

Fig. B-2

Values of k and j for various percentages of reinforcement

The cross-section of a beam may be determined when M, f_s, f_m and p are known or assumed. If the value of p is less than that given by equation (8), then the required cross-section of the beam is:

$$bd^2 = \frac{M}{f_s pj} \quad \ldots\ldots (9)$$

If the value of p is greater than that given by equation (8), then the required cross-section of the beam is:

$$bd^2 = \frac{M}{\frac{1}{2} f_m kj} \quad \ldots\ldots (10)$$

The values k and j can be obtained by using equations (4) and (5), or more readily from Fig. B-2.

It has been found convenient to substitute the letter K for the denominator in equations (9) and (10). The symbol K may properly be called the coefficient of resistance of the steel and brickwork, respectively, and equations (9) and (10) may be rewritten as:

$$bd^2 = M_s/K_s \ldots\ldots (9a), \text{ and}$$
$$bd^2 = M_m/K_m \ldots\ldots (10a)$$

Since for balanced design, $M_s = M_m$, then $K_s = K_m$, and $K = M/bd^2$, or

$$bd^2 = M/K \ldots\ldots (11)$$

For values of K, see Tables B-2 to B-7.

B-4. SHEARING STRESSES

It has been found that shear is the principal factor in the failure of reinforced concrete beams by diagonal tension and should not be overlooked in reinforced brick masonry. In theory, bond stress is also a function of shear. The commonly accepted assumption is that shearing stress is the same at all points between the reinforcing steel and the neutral axis. The intensity of the horizontal shearing stress at any point is equal to that of the vertical shearing stress. Above the neutral axis, the shear decreases according to the parabolic law as in a homogeneous rectangular beam, becoming zero at the upper surface. The standard equation for the intensity of the shearing stress on any plane between the reinforcement and neutral axis is:

$$v = \frac{V}{bjd} \ldots\ldots (12)$$

where v = the unit shearing stress

and V = the total shear

b, j and d are the same as given in Section B-2.

For approximate computations, the value of j may be taken as $7/8$. The equation then becomes:

$$v = \frac{8V}{7bd} \ldots\ldots (13)$$

B-5 BOND STRESSES

The tension in the horizontal reinforcement near the lower surface of an R-B-M (reinforced brick masonry) beam is maximum at the point of maximum moment. In a simple beam this is usually near the center, and decreases toward the supports. The difference in tension between any two points is transmitted from the steel to the brick masonry by the bond between the steel, mortar and brick. The equation for bond stress, in beams with horizontal reinforcement, is:

$$u = \frac{V}{\Sigma ojd} \ldots\ldots (14)$$

where u = bond stress per unit area.

Σo = sum of perimeters of all bars at a given section.

It is readily seen that the bond stress varies directly with the total shear (V) and inversely with the perimeters of the reinforcing bars (Σo). Shear diagrams may be used to represent the variation of bond stress along a beam. Approximations may be made by using the average value of j as $\frac{7}{8}$.

For a more complete discussion of the theory of shear and bond stress, the reader is referred to any standard text book on strength of materials.

B-6. APPLICATION OF FORMULAE AND TABLES

Illustrative Problem No. 1

Design an R-B-M slab for a span of 5 ft. 6 in. to support a uniform load of 50 lb. per sq. ft. Brick prism test specimens have shown an average ultimate compressive strength (f'_m) of 2200 psi; $f_s = 18{,}000$ psi.

Solution: The ultimate strength ($f'_m = 2200$) of the prisms indicates that, in the design, a safe working stress of $2200/3 = 733$ psi may be used for the unit compressive strength of brick masonry. However, for convenience in using the tables, the next lower value listed in the tables ($f_m = 700$) will be used. The use of this lower value will increase the safety factor.

The following values will therefore be used as a basis for design (see Section 304):

$$\begin{aligned} f_m &= 700 \text{ psi} \\ v_m &= 50 \text{ psi} \\ u &= 80 \text{ psi} \\ n &= 15 \\ f_s &= 18{,}000 \text{ psi} \end{aligned}$$

The weight of the brick masonry is assumed to be 125 lb. per cu. ft.

Assume the thickness of slab as $2\frac{1}{4}$ in. (brick laid flat).

The dead load of the slab $= \dfrac{125 \times 2\frac{1}{4}}{12} = 23\frac{1}{2}$ lb. per sq. ft. Use 30 lb. to allow for $\frac{1}{2}$ in. of plaster on under side. The total load (L.L. + D.L.) $= 50 + 30 = 80$ lb. per sq. ft.

Designing the slab as a simple beam, 1 ft. wide, ($b = 12$ in.) the maximum bending moment, $M = \dfrac{Wl^2}{8} = \dfrac{80 \times 5\frac{1}{2} \times 5\frac{1}{2} \times 12}{8} = 3{,}630$ in.-lb.

Using equation (11) from Section B-3:

$$\begin{aligned} bd^2 &= M/K \\ b &= 12 \text{ in.} \\ M &= 3630 \text{ in.-lb.} \\ K &= 113.1 \text{ (See Table B-5)} \end{aligned}$$

Therefore,

$$d^2 = \frac{M}{bK} = \frac{3630}{12 \times 113.1} = 2.68$$

$$d = \sqrt{2.68} = 1.64 \text{ in.}$$

with a thickness of brick equal to $2\frac{1}{4}$ in., the distance from bottom to center of steel is $2.25 - 1.64 = 0.61$ in., almost $\frac{5}{8}$ in. A half inch is permissible when the bottom of the slab is protected with not less than $\frac{1}{2}$ in. of plaster.

Referring again to Table B-5, a balanced design, when $n = 15$, $f_m = 700$ and $f_s = 18{,}000$, should have the following values:

$p = 0.0072$
$k = 0.3684$
$j = 0.8772$

The required area of steel, $A_s = pbd = 0.0072 \times 12 \times 1.64 = 0.1414$ sq. in. Equation (6) may be written in the form: $A_s = M/f_sjd$, and used as a check.

$$A_s = \frac{3630}{18{,}000 \times 0.877 \times 1.64} = 0.1401 \text{ sq. in.}$$

By computation, it is found that No. 2 round bars (see Table B-1) in ½-in. mortar joints between brick laid flat (3½ in. + ½ in. = 4-in. centers) have an area of steel per foot width of slab equal to 0.15 sq. in. The actual effective depth d that is available is 2¼ in. − ½ in. = 1¾ in. or 1.75 in., instead of the 1.64 in. calculated as the minimum required.

Using d as 1.75 in. in equation (6):

$$A_s = \frac{3630}{18{,}000 \times 0.877 \times 1.75} = 0.1314 \text{ sq. in.}$$

Therefore, 0.15 sq. in. of steel per ft. of slab width will be more than sufficient.

The design will now be analyzed for the values of f_s and f_m, with $A_s = 0.15$, $b = 12$ in. and $d = 1.75$ in.

$$p = \frac{A_s}{bd} = \frac{0.15}{12 \times 1.75} = 0.0072$$

From Fig. B-2, when $p = 0.0072$ and $n = 15$, $k = 0.368$ and $j = 0.877$.
Substituting in equation (6):

$$f_s = \frac{M}{A_sjd} = \frac{3630}{0.15 \times 0.877 \times 1.75} = 15{,}800 \text{ psi.}$$

The compressive stress in brick masonry is checked by substituting in equation (7):

$$f_m = \frac{2pf_s}{k} = \frac{2 \times 0.0072 \times 15{,}800}{0.368} = 620 \text{ psi.}$$

The calculated unit stresses in the reinforcement and brickwork are less than the allowable unit stresses. The design is satisfactory from the standpoint of compressive stress in the brick masonry (f_m) and tensile stress in the steel (f_s), but will be checked for shear and bond stresses.

The total shear (V), at the support, is:

$$V = \frac{wl}{2} = \frac{80 \times 5\frac{1}{2}}{2} = 220 \text{ lb.}$$

Unit shearing stress (v), using equation (12):

$$v = \frac{V}{bjd} = \frac{220}{12 \times 0.877 \times 1.75} = 12 \text{ psi.}$$

The allowable unit shearing stress for the brick masonry in this problem is 50 psi No stirrups or bent-up bars will be necessary.

The unit bond stress (u) is determined by using equation (14).

$$u = \frac{V}{\Sigma ojd}$$

The summation of perimeters (Σo) is determined first with brick laid flat and ¼-in. round bars in ½-in. mortar joints. The centering of the bars is 3½ in. + ½ in. = 4 in.

The total perimeters per 12-in. width of slab is $\frac{0.786 \times 12}{4} = 2.36$ in.

Another method is to multiply the perimeter of one bar by the area of steel (A_s) per foot width and divide by the area of one bar:

$$\Sigma o = \frac{0.786 \times 0.15}{0.05} = 2.36 \text{ in.}$$

Substituting in equation (14):

$$u = \frac{V}{\Sigma ojd} = \frac{220}{2.36 \times 0.877 \times 1.75} = 60.8 \text{ psi.}$$

The allowable unit bond stress in this problem is 80 psi, and it will not be necessary to hook the bars or provide special anchorage.

Therefore, the design satisfies all conditions and will consist of brick laid flat with ¼-in. round bars in each ½-in. mortar joint and placed 1¾ in. below the top surface or ½ in. above bottom surface of the brick. The under side of the slab is to be covered with plaster ½ in. thick.

Illustrative Problem No. 2

A slab is made with brick on edge (3¾ in. deep) in ½-in. mortar joints. The ¼-in. round reinforcing rods are placed in each joint and 3 in. below the surface. The spacing of the bars is, therefore, 2¼ in. + ½ in. = 2¾ in. Test prisms of the brickwork indicate an ultimate compressive strength (f'_m) of 1840 psi. What is the safe uniform live load per square foot of slab on a span of 8 ft.? The allowable tensile stress in the reinforcing steel (f_s) is 18,000 psi.

Solution: The slab will be analyzed as a simple beam 12 in. wide. The resisting moment (M) will be determined for both the steel and brick masonry. The lesser of the two will be used to calculate the allowable load.

The ultimate strength ($f'_m = 1840$) of the test prisms will permit the use of the following values:

$f_m = 600$ psi
$v_m = 50$ psi
$u = 80$ psi
$n = 18$

Unless otherwise specified, the weight of the masonry is assumed to be 125 lb. per cu. ft. The dead load per square foot is, therefore, $125 \times \frac{3\frac{3}{4}}{12} = 39$ lb.

The reinforcement, No. 2 bars on 2¾-in. centers, makes the area of steel (A_s) per foot width:

$$A_s = \frac{0.05 \times 12}{2\frac{3}{4}} = 0.218 \text{ sq. in.}$$

$$p = \frac{A_s}{bd} = \frac{0.218}{12 \times 3} = 0.0061$$

and the summation of perimeters (Σo) per foot width:

$$\Sigma o = \frac{0.786 \times 12}{2\frac{3}{4}} = 3.43 \text{ in.}$$

From Table B-4, when $n = 18$, $f_m = 600$ and $f_c = 18{,}000$; $p = .0062$, $k = 0.375$, $j = 0.875$ and $K = 98.44$.

The maximum moment for the reinforcing steel:

$$M_s = jA_sf_sd = 0.875 \times 0.218 \times 18{,}000 \times 3 = 10{,}300 \text{ in.-lb.}$$

The maximum moment for the brick masonry:

$$M_m = \tfrac{1}{2} f_m kjbd^2 = \tfrac{1}{2} \times 600 \times 0.375 \times 0.875 \times 12 \times 3^2 = 10{,}600 \text{ in.-lb.}$$

The smaller moment of 10,300 in.-lb. (860 ft.-lb.) will be used and, since the maximum moment in a simple beam with uniform load (w) is

$$M = \frac{wl^2}{8}$$

$$w = \frac{8M}{l^2} = \frac{8 \times 860}{8 \times 8} = 107 \text{ lb. per sq. ft.}$$

The dead load has been calculated to be 39 lb. per sq. ft.; therefore, the allowable live load will be 107 − 39 = 68 lb. per sq. ft.

The slab will be checked to see whether or not this loading will cause excessive stresses in shear or bond.

The total shear:

$$V = \frac{wl}{2} = \frac{107 \times 8}{2} = 428 \text{ lb.}$$

Unit shearing stress:

$$v = \frac{V}{bjd} = \frac{428}{12 \times 0.875 \times 3} = 13.6 \text{ psi.}$$

The allowable shearing stress is 50 psi.

Unit bond stress:

$$u = \frac{V}{\Sigma ojd} = \frac{428}{3.43 \times 0.875 \times 3} = 47.5 \text{ psi.}$$

The allowable bond stress is 80 psi; therefore, the R-B-M slab described in this problem will safely support a uniform live load of 68 lb. per sq. ft.

Illustrative Problem No. 3

Design an R-B-M beam with a span of 12 ft. to support a live load of 1450 lb. per lin. ft. Use $f_m = 600$, $f_s = 18{,}000$ and $n = 18$.

Solution: The total live load is 1450 lb. per ft. of beam. Assuming the dead load of the beam to be 150 lb. per ft., the total dead and live load per ft. of beam will be $1450 + 150 = 1600$ lb.

From Table B-4, we find: $p = 0.0062$, $k = 0.375$, $j = 0.875$, and $K = 98.44$.

$$M = \frac{wl^2}{8} = \frac{1600 \times 12^2 \times 12}{8} = 346{,}000 \text{ in.-lb.}$$

For a balanced design, equation (11) gives:

$$bd^2 = \frac{M}{K} = \frac{346{,}000}{98.44} = 3520$$

Assume b as 8 in., then $d^2 = \frac{3520}{8} = 440$; $d = 21.0$ in.

$$A_s = pbd = 0.0062 \times 8 \times 21.0 = 1.04 \text{ sq. in.}$$

Use 4 No. 5 bars = 1.24 sq. in. Four bars on $1\frac{1}{4}$-in. spacing, ct. to ct., require $3 \times 1\frac{1}{4}$ in. $= 3\frac{3}{4}$ in.; $3\frac{3}{4}$ in. $+ \frac{5}{8}$ in. $= 4\frac{3}{8}$ in. out to out of bars. This will allow $\frac{8 - 4\frac{3}{8}}{2} = 1\frac{13}{16}$ in. cover at sides of beams. Eight courses of brick @ $2\frac{2}{3}$ in. $= 21\frac{1}{3}$ in.

The minimum effective depth required was 21.0 in.; therefore, a depth of 21.3 in. is satisfactory. The total depth of beam is:

Effective depth to center of steel	= 21.3 in.
Soffit course of brick	= 2.41 in.
Total overall depth	23.71 in.

$$\text{Weight of beam} = \frac{125 \times 8 \times 23.7}{12 \times 12} = 165 \text{ lb. per lin. ft.}$$

Since the calculated weight is fairly close to the assumed weight, many designers would accept the design. However a recheck will be made.

The total dead load plus live load is $1450 + 165 = 1615$ lb. per lin. ft.

$$M = \frac{wl^2}{8} = \frac{1615 \times 12^2 \times 12}{8} = 355{,}000 \text{ in.-lb.}$$

$$d^2 = \frac{M}{bK} = \frac{355{,}000}{8 \times 98.44} = 450.0; \ d = 21.2 \text{ in.}$$

$$A_s = pbd = 0.0062 \times 8 \times 21.2 = 1.05 \text{ sq. in.}$$

The beam will now be checked for shearing stress.

$$V = \frac{wl}{2} = \frac{1615 \times 12}{2} = 9700 \text{ lb.}$$

$$v = \frac{V}{bjd} = \frac{9700}{8 \times 0.875 \times 21.2} = 65.0 \text{ psi}$$

Allowable shear on brick was	50.0 psi
Shear to be taken by stirrups	15.0 psi

Try ¼-in. round Z bar. The required spacing of stirrups at the supports:

$$s = \frac{A_v \times f_v}{vb} = \frac{0.05 \times 18{,}000}{15.0 \times 8} = 7.5 \text{ in.}$$

The first stirrup will be placed not more than ½ × 7.5 in., or 3 in. from the beam support.

No stirrups will be required at $\frac{12}{2} \times \frac{50}{65} = 4.6$ ft. from the center of the span.

Therefore, stirrups will be required up to 1.4 ft. or 1 ft. 5 in. from the ends of the beam.

If the beam is built with all stretcher courses, the stirrups may be spaced exactly as calculated. As in reinforced concrete design, a shear diagram may be used to determine stirrup spacing. However, this spacing can be approximated with a reasonable degree of accuracy. For instance, midway between the support and that point on the beam where no stirrups are required, the stirrup spacing is double that required at the support. However, the maximum spacing should be approximately ½ the effective depth (d/2). In this problem, therefore, we shall locate the ¼-in. Z bar stirrups at 3, 10 and 17 in. from the support.

The unit bond stress:

$$u = \frac{V}{\Sigma ojd} = \frac{9700}{7.86 \times 0.875 \times 21.2} = 66.5 \text{ psi.}$$

The allowable bond stress for deformed bars is 160 psi. No special anchorage will be required.

TABLE B-1

AREAS, PERIMETERS AND WEIGHTS OF BARS

Standard A305 Reinforcing Bars

Bar Sizes			Nominal Dimensions—Round Sections		
Old, in.	New, No.	Weight, lb. per ft.	Diameter, in.	Cross-Sectional Area, sq. in.	Perimeter, in.
¼	2	.167	.250	.05	.786
⅜	3	.376	.375	.11	1.178
½	4	.668	.500	.20	1.571
⅝	5	1.043	.625	.31	1.963
¾	6	1.502	.750	.44	2.356
⅞	7	2.044	.875	.60	2.749
1	8	2.670	1.000	.79	3.142

TABLE B-2

VALUES OF p, k, j AND K FOR VARIOUS COMBINATIONS OF STEEL AND BRICK STRESSES FOR RECTANGULAR BEAMS AND SLABS

$$k = \frac{1}{1 + \frac{f_s}{nf_m}} \qquad p = \frac{f_m k}{2f_s} \qquad j = 1 - \frac{k}{3} \qquad K = \frac{1}{2} f_m kj \text{ or } pf_s j.$$

n = 25

$f_s = 18,000$

f_m	p	k	j	K
400	.0040	.3571	.8810	62.92
450	.0048	.3846	.8718	75.44
500	.0057	.4098	.8634	88.46
550	.0066	.4331	.8556	101.90
600	.0076	.4545	.8485	115.69
650	.0086	.4745	.8418	129.81
700	.0096	.4930	.8357	144.20
750	.0106	.5102	.8300	158.80
800	.0117	.5263	.8246	173.60
900	.0139	.5556	.8148	203.60
1000	.0161	.5814	.8062	234.36
1100	.0184	.6044	.7985	265.44
1200	.0208	.6250	.7917	296.89
1300	.0232	.6436	.7855	328.61

$f_s = 20,000$

f_m	p	k	j	K
400	.0033	.3333	.8889	59.26
450	.0041	.3601	.8800	71.28
500	.0048	.3846	.8718	83.82
550	.0056	.4074	.8642	96.82
600	.0064	.4286	.8571	110.21
650	.0073	.4483	.8506	123.93
700	.0082	.4666	.8445	137.92
750	.0091	.4838	.8387	152.17
800	.0100	.5000	.8333	166.67
900	.0119	.5294	.8235	196.18
1000	.0139	.5556	.8148	203.60
1100	.0159	.5789	.8070	256.95
1200	.0180	.6000	.8000	288.00
1300	.0215	.6599	.7800	334.62

TABLE B-3

VALUES OF p, k, j AND K FOR VARIOUS COMBINATIONS OF STEEL AND BRICK STRESSES FOR RECTANGULAR BEAMS AND SLABS

$n = 20$

$f_s = 18{,}000$

f_m	p	k	j	K
450	.0042	.3333	.8889	66.67
500	.0050	.3571	.8810	78.65
550	.0058	.3794	.8735	91.14
600	.0067	.4000	.8667	104.00
650	.0076	.4193	.8602	117.22
700	.0085	.4374	.8542	130.77
750	.0095	.4545	.8485	144.62
800	.0105	.4706	.8431	158.71
900	.0125	.5000	.8333	187.50
1000	.0146	.5263	.8246	217.00
1100	.0168	.5500	.8167	247.05
1200	.0190	.5714	.8095	277.53
1300	.0213	.5910	.8030	308.47
1400	.0237	.6086	.7971	339.58

$f_s = 20{,}000$

f_m	p	k	j	K
450	.0035	.3103	.8966	62.60
500	.0042	.3333	.8899	74.07
550	.0049	.3548	.8817	86.06
600	.0056	.3750	.8750	98.44
650	.0064	.3939	.8687	111.25
700	.0072	.4117	.8628	124.33
750	.0080	.4286	.8571	137.75
800	.0089	.4444	.8519	151.45
900	.0107	.4737	.8421	179.48
1000	.0125	.5000	.8333	208.25
1100	.0144	.5238	.8254	237.67
1200	.0164	.5454	.8182	267.68
1300	.0184	.5652	.8116	297.95
1400	.0204	.5834	.8055	328.93

TABLE B-4

VALUES OF p, k, j AND K FOR VARIOUS COMBINATIONS OF STEEL AND BRICK STRESSES FOR RECTANGULAR BEAMS AND SLABS

$n = 18$

$f_s = 18{,}000$

f_m	p	k	j	K
500	.0046	.3333	.8890	74.07
550	.0054	.3548	.8820	86.02
600	.0062	.3750	.8750	98.44
650	.0071	.3939	.8690	111.25
700	.0080	.4117	.8630	124.33
750	.0089	.4286	.8571	137.75
800	.0099	.4444	.8520	151.45
900	.0118	.4737	.8420	179.48
1000	.0139	.5000	.8330	208.25
1100	.0169	.5238	.8250	237.67
1200	.0182	.5454	.8180	267.68
1300	.0204	.5652	.8110	297.95
1400	.0227	.5833	.8056	328.93
1500	.0250	.6000	.8000	360.00

$f_s = 20{,}000$

f_m	p	k	j	K
500	.0039	.3103	.8966	69.55
550	.0046	.3311	.8896	81.00
600	.0053	.3506	.8831	92.88
650	.0060	.3691	.8770	105.20
700	.0068	.3865	.8712	117.86
750	.0076	.4030	.8657	130.82
800	.0084	.4186	.8605	144.08
900	.0101	.4475	.8508	171.33
1000	.0118	.4737	.8421	199.45
1100	.0138	.5000	.8333	229.17
1200	.0156	.5192	.8269	257.60
1300	.0175	.5392	.8203	287.50
1400	.0195	.5576	.8141	317.76
1500	.0215	.5745	.8085	348.37

TABLE B-5

VALUES OF p, k, j AND K FOR VARIOUS COMBINATIONS OF STEEL AND BRICK STRESSES FOR RECTANGULAR BEAMS AND SLABS

$n = 15$

$f_s = 18,000$

f_m	p	k	j	K
500	.0041	.2941	.9020	66.32
550	.0048	.3143	.8952	77.37
600	.0056	.3333	.8889	88.88
650	.0063	.3514	.8829	100.80
700	.0072	.3684	.8772	113.10
750	.0080	.3846	.8718	125.70
800	.0089	.4000	.8667	138.70
850	.0098	.4146	.8618	151.90
900	.0107	.4288	.8571	165.40
950	.0117	.4419	.8527	179.00
1000	.0126	.4545	.8485	192.80
1100	.0146	.4783	.8406	221.10
1200	.0167	.5000	.8333	250.00
1300	.0188	.5200	.8267	279.40
1400	.0209	.5385	.8205	309.30
1500	.0232	.5556	.8148	339.50

$f_s = 20,000$

f_m	p	k	j	K
500	.0034	.2727	.9091	61.98
550	.0040	.2920	.9027	72.49
600	.0047	.3103	.8966	83.46
650	.0053	.3277	.8908	94.87
700	.0060	.3443	.8852	106.70
750	.0068	.3600	.8800	118.80
800	.0075	.3750	.8750	131.30
850	.0083	.3893	.8702	144.00
900	.0091	.4030	.8657	157.00
950	.0099	.4161	.8613	170.20
1000	.0107	.4286	.8571	183.70
1100	.0124	.4521	.8493	211.20
1200	.0142	.4737	.8421	239.30
1300	.0160	.4937	.8354	268.10
1400	.0179	.5122	.8293	297.30
1500	.0199	.5294	.8235	327.00

TABLE B-6

VALUES OF p, k, j AND K FOR VARIOUS COMBINATIONS OF STEEL AND BRICK STRESSES FOR RECTANGULAR BEAMS AND SLABS

$n = 12$

$f_s = 18,000$

f_m	p	k	j	K
800	.0077	.3478	.8841	123.00
900	.0094	.3750	.8750	147.70
1000	.0111	.4000	.8667	173.30
1100	.0129	.4230	.8590	199.90
1125	.0134	.4286	.8571	206.60
1200	.0148	.4444	.8519	227.20
1300	.0168	.4643	.8452	255.10
1400	.0188	.4828	.8391	283.60
1500	.0208	.5000	.8333	312.50

$f_s = 20,000$

f_m	p	k	j	K
800	.0065	.3243	.8919	115.70
900	.0079	.3506	.8831	139.30
1000	.0094	.3750	.8750	164.10
1100	.0109	.3976	.8675	189.70
1125	.0113	.4030	.8657	196.20
1200	.0126	.4186	.8605	216.10
1300	.0142	.4382	.8539	243.20
1400	.0160	.4565	.8478	270.90
1500	.0178	.4737	.8421	299.20

TABLE B-7

VALUES OF p, k, j AND K FOR VARIOUS COMBINATIONS OF STEEL AND BRICK STRESSES FOR RECTANGULAR BEAMS AND SLABS

$n = 10$

$f_s = 18{,}000$

f_m	p	k	j	K
1000	.0099	.3571	.8810	157.3
1100	.0116	.3793	.8736	182.3
1200	.0133	.4000	.8667	208.0
1300	.0151	.4193	.8602	234.4
1350	.0161	.4286	.8571	248.0
1400	.0170	.4374	.8542	261.5
1500	.0189	.4545	.8485	289.2

$f_s = 20{,}000$

f_m	p	k	j	K
1000	.0083	.3333	.8889	148.1
1100	.0098	.3548	.8817	172.1
1200	.0113	.3750	.8750	196.9
1300	.0128	.3939	.8687	222.4
1350	.0136	.4030	.8657	235.5
1400	.0144	.4117	.8628	248.7
1500	.0161	.4286	.8571	275.5

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